FLOOD PLAN COORDINATION ORGANIZATION (FPCO)

KREDITANSTALT FÜR WIEDERAUFBAU (KfW)

CAISSE FRANCAISE DE DEVELOPPEMENT (CFD)

BANK PROTECTION AND RIVER TRAINING (AFPM) PILOT PROJECT FAP 21/22

DRAFT FINAL REPORT PLANNING STUDY

VOLUME II

ANNEX 1 : River Training and Morphological Response
ANNEX 2 : River Morphology

DECEMBER 1992

CONSULTING CONSORTIUM FAP 21/22

RHEIN-RUHR ING.-GES.MBH, DORTMUND/GERMANY
COMPAGNIE NATIONALE DU RHONE, LYON/FRANCE
PROF.DR. LACKNER&PARTNERS, BREMEN/GERMANY
DELFT HYDRAULICS, DELFT/NETHERLANDS

In association with:
BANGLADESH ENGINEERING & TECHNOLOGICAL SERVICES LTD.(BETS)
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CONTENTS OF FINAL REPORT

*) to be submitted with the FAP 21 draft final report

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BANK PROTECTION AND RIVER TRAINING (AFPM) PILOT PROJECT FAP 21/22

DRAFT FINAL REPORT

ANNEX 1

RIVER TRAINING AND MORPHOLOGICAL RESPONSE

DECEMBER 1992

ANNEX 1

RIVER TRAINING AND MORPHOLOGICAL RESPONSE

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LIST OF SYMBOLS

B	=	Channel width	(m)
B_b	=	Bankfull width	(m)
B_t	=	Exponent of power-law sediment transport formula	
C	=	Chézy's roughness parameter	($m^{1/2}/s$)
D	=	Bed material size	(mm)
E	=	Annual erosion	(mm)
f	=	Lacey's silt factor	(-)
$f(\theta_0)$	=	Cross-sectional average of function for the influence of gravity pull along a transverse bed slope	(-)
G	=	Erodibility co-efficient	(-)
g	=	Acceleration due to gravity	(m/s^2)
H	=	Outer bank height	(m)
H_d	=	Draft of screen	(m)
H_p	=	Height of screen	(m)
H_{fb}	=	Free board of bank above water level	(m)
h	=	Average water depth	(m)
h_0	=	Cross-sectional average of water depth	(m)
h_b	=	Bankfull depth	(m)
h_w	=	Water depth near bank	(m)

h_{wer}	=	Critical water depth near bank	(m)
i	=	River slope	(-)
i_b	=	River slope at bankfull stage	(-)
i_v	=	Valley slope	(-)
k	=	Streamwise wave number	
L	=	Length of screen	(m)
m	=	Transverse mode, indicating the number of channels per cross-section	(-)
n	=	q_s/S_e, $\Delta Q/Q$, $\Delta S/S$	(-)
p	=	Sinuosity	(-)
Q	=	River discharge	(m^3/s)
Q_b	=	Bankfull discharge	(m^3/s)
R	=	Radious of curvature of bend	(m)
R_h	=	Hydraulic radious	(m)
S	=	Sediment transport	(m^3/s)
S_b	=	Sediment transport at bankfull stage	(m^3/s)
T	=	Time	
u	=	Flow velocity	(m/s)
u_b	=	Flow velocity at bankfull stage	(m/s)
u_w	=	Flow velocity near bank	(m/s)
V	=	Sediment transport integrated over the year	
W_{wer}	=	Critical flow velocity	(m/s)

z	=	Side slope	(-)
z_b	=	Bed level	(m)
α	=	Angle of attack	(degree)
η	=	Ratio of water depths	(-)
λ	=	Cut-off ratio	(-)
ν	=	Sediment distribution parameter	(-)
ξ	=	Co-efficient for the effect of streamwise bed slope	(-)
π	=	3.14159	
ρ	=	Density of water	(kg/m^3)
Ω	=	Stream power	(N/s)
φ	=	Bank slope	(-)

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FOREWORD

FOREWORD

(i) REFERENCE

This report on the river training and morphological response was prepared as Technical Report No.2 of the Bank Protection and River Training/Active Flood Plain Management (AFPM) Pilot Project FAP 21/22.

(ii) OBJECTIVES

The main objectives of this report (see Tasks 26 and 27 as per Consultancy Agreement) are:

- o to prepare a "state-of-the-art" and critical review of possible river training measures for major braided rivers both of a permanent and a recurrent nature, and
- o to prepare a "state-of-the-art" and critical review and to select prediction methods for the morphological response and strategies (see definition hereafter).

Other objectives (parts of Tasks 28 through 30 as per Consultancy Agreement) are :

- o to formulate active flood plain management (AFPM) measures and strategies defining the scope of river training in this report (see Chapter 1), and
- o to assess the effectivity of the various measures.

(iii) DEFINITIONS

In order to avoid confusion about the meaning of various key-words used in this report, the following definitions are given:

- o **measures**
  River training measures consist of both construction of structures like revetments, groynes, vanes, etc. and activities such as dredging. A measure is often considered 'stand-alone' and effects are often considered near-field (short river section) short-term (1 to 2 year)

- o **strategies**
  Strategies are a combination of measures designed to reach all objectives formulated as part of the strategy. In general the timing of construction and the hydraulic and morphological effects play an important role. A strategy is often considered far-field (substantial part of the river) where both short and long term (10 to 20 years) effects are important

- **permanent/recurrent**
  In literature sometimes the terminology permanent, semi-permanent and temporal measures are used for periods upto 50 years (engineering time scale), 5 years and 1 season, respectively. The last two categories are called here recurrent.

Aspects of measures and strategies are, amongst others, accuracy of prediction of effects, effectivity, reliability, sustainability, cost-effectivity, structural design, material, construction, etc. Some of these aspects are defined below :

- **effectivity**
  The effectivity expresses in how far and how fast hydraulic and/or morphological targets can be achieved

- **reliability**
  The reliability is used in connection with either the realisation of the measures (construction of structures, dredging) or the prediction of morphological effects

- **sustainability**
  The sustainability means the effectivity as a function of time
  Note : the sustainability of the measures or works (stability, life-time, etc.) is expressed here by the terms permanent and recurrent

- **cost-effectivity**
  The cost-effectivity is the relation between the total cost and the effectivity of a measure

- **low cost measure**
  Part of recurrent measures are called low cost measures if the costs are low in comparison with the nearest alternative. Compare for instance cutter suction dredging versus water injection dredging and cutter suction dredging versus bandalling. In both cases the alternative for cutter suction dredging may appear to be the low cost measure.

(iv) SET UP OF REPORT

According to the tasks 26 through 30 as per Consultancy Agreement this ANNEX 1 consists of three (interrelated) parts:

A. <u>"State-of-the-Art" in River training</u>, summarizing and reviewing possible river training measures and strategies. Both permanent and recurrent measures with respect to the braiding character of the Jamuna river will be considered.

B. Preliminary Selection of Recurrent Measures ; this preliminary selection has been based on the hydraulic and morphological effects only.

C. Morphological Response to Measures and Strategies ; an introduction to a quantitative assessment of the morphological response to the individual measures will be presented. Next to this the available predication methods for describing the behaviour of a river system will be presented briefly.

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PART A

"STATE-OF-THE-ART" IN RIVER TRAINING

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PART A: "STATE-OF-THE-ART" IN RIVER TRAINING

1 INTRODUCTION

1.1 OBJECTIVE

The "State-of-the-Art" on River Training has to be considered as an inventory and critical review of river training measures aiming at their possible applicability in the major braided Jamuna river. Both permanent and recurrent measures will be reviewed.

1.2 LIMITATIONS AND SET UP

In order to avoid rewriting of vast manuals on river training, the following limitations have been applied:

- o Recurrent measures are emphasized. The permanent measures are presented with a lower degree of detail as this subject will be further elaborated during the design phase of the permanent pilot structures of FAP 21
- o The report is focussing on river training for erosion control, especially the control of the outer banks of the outer channels, being a critical part of AFPM, (see further next section). Note that also the control of stable chars and land reclamation might be of particular interest with respect to AFPM.
- o Measures and strategies are dealt with in respect to the technical feasibility, where principles are more important than structural details. Example: the principle how sedimentation can be generated using a vane is described, rather than how the vane can be constructed as a recurrent or as a more permanent structure.

In brief the explanation of the contents of the "State-of-the-Art" read as follows:

- o After the general description of the principles of active flood plain management, the most important aspects for the Jamuna are discussed, to limit the scope of river training, (see Chapter 2)
- o Thereafter follows a description of existing measures, both permanent and recurrent ones, (see Chapter 3).
- o A description of existing strategies is given in Chapter 4. Also the results of the study tour to China (Yellow River and Yangtze Kiang) and the United States (Mississippi River) will be presented in this chapter, including some final conclusion on river training and strategies.

2 ACTIVE FLOOD PLAIN MANAGEMENT (AFPM)

2.1 TARGET OF RIVER TRAINING

In general it can be stated that river training measures are all engineering works in a river to regulate the flow of water and sediments in a river (thus controlling to a certain extent discharges, water levels and erosion and sedimentation of the river bed) for the sake of irrigation, hydropower, navigation and other use of the river.

This wide scope may be limited in the context of the objectives of FAP 21/22 as follows :

- o type of river : major braided sand bed river
- o main target of river training : erosion control
- o main area of interest : the flood plain and the management thereof (AFPM). As a consequence of the last two points this report is focussing upon
- o erosion control of the outer banks of the outer channels.

The last points are elaborated hereafter.

2.2 CONTEXT OF AFPM

A typical cross-section of a major braided river is schematized as follows (see Fig. 2.2-1).

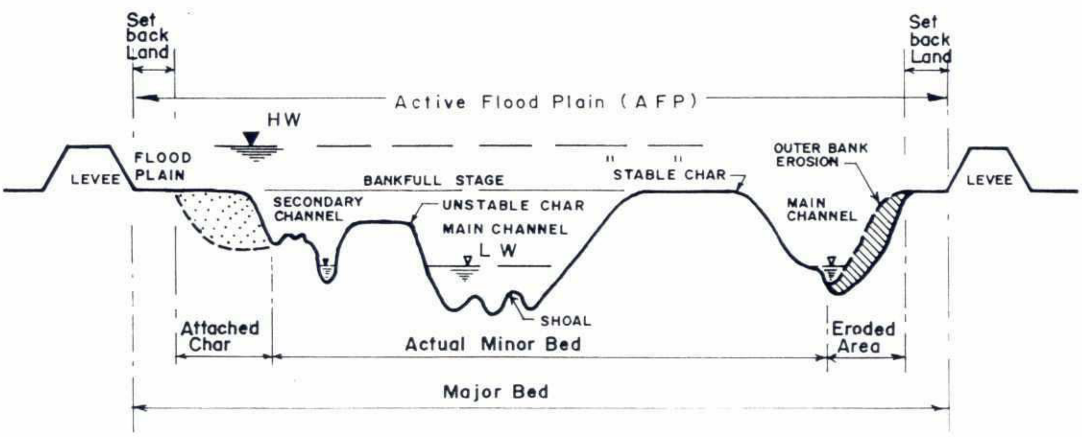

Fig. 2.2-1: Typical cross-section of a braided river

Note : This schematization is based upon the typical cross-sections of the Jamuna River. An example is given in Fig. 2.2-2.

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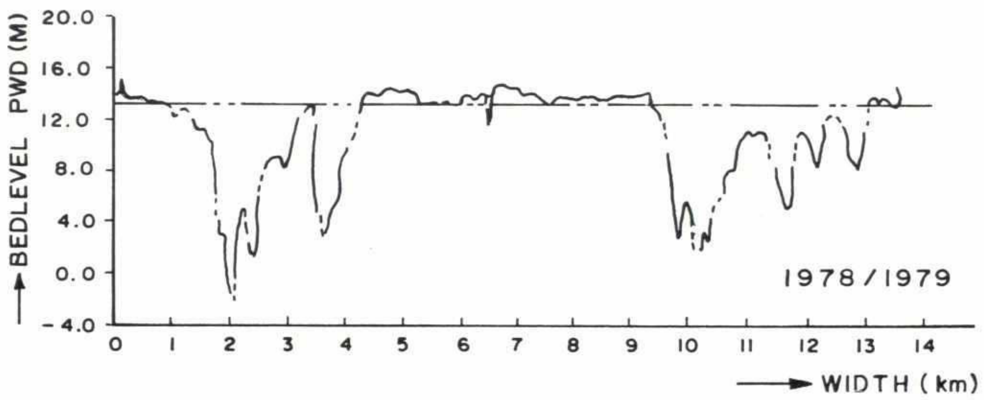

Fig. 2.2-2: Typical cross-section Jamuna river (RPT 1989)

The main characteristics of this cross-section are :

o **levee**
The embankments or levees can be natural or man-made, crest-levels often around flood level; horizontal position depending on land use at both sides and outer channel migration

o **flood plain**
Vegetated plain, vertically more or less stable, level depends on the use (brush-vegetation promotes sedimentation resulting in higher levels). Due to outer bank erosion the flood plain may change

o **main and secondary channels**
An arbitrary split of the cross-section into relative big and relative small channels respectively

o **unstable char**
A bare sandbank with crest levels between the low water (LW) level and bankfull stage

o **shoal**
A sandbank below the LW level

o **"stable" char**
A stable char is in fact a part of the flood plain. The char is vegetated and at the flood plain level

o **actual minor bed**
Width taken by the total of main and secondary channels and the stable and unstable chars at a certain moment

2a

- **active flood plain (AFP)**
  Total area that potentially can be eroded by the river channels over a long period due to the shifting of the minor bed, hence including the minor bed and the attached chars

- **attached chars**
  Area between the edge of the active floodplain and the actual location of bank of the minor bed, potentially previously eroded or to be eroded by shifts of the minor bed within the active flood plain

- **set-back land**
  Distance between the levee and the edge of the active flood plain (the horizontal safety can become negative if the levee is eroded by a river channel)

- **major bed**
  The river bed between the two levees, including the floodplain, attached chars and minor bed; in the case of erosion of one of the levees also a part of the land previously protected by this levee becomes part of the major bed. This area is defined again when a retired embankment has been constructed. If no levee is present an imaginary edge of the major bed has to be defined based on an assessed boundary between river flooding and flooding due to excessive rainfall.

Considering the general cross-section, the control of the river can be split up into amongst others:

- **flood control**
  In a strict sense especially dealing with the management of the levels. Especially the flood flow conditions are of importance

- **flood plain control**
  Dealing with the management of the flood plain including AFPM. Especially the conditions around bank full flow are relevant, (see further next section)

- **fairway control**
  Dealing with the management of the navigable channels especially during low flow conditions.

Notes :

1. As long as levees are not that high (in the Jamuna river up to 5 m above the flood plain level), erosion control of levees is relatively easy as long as the levees are protected by flood plains.
2. To guarantee the protection of levees the erosion of the flood plains should be controlled and therefore the AFPM is emphasized in the following sections.

3. Fairway control seems less important in the context of FAP 21/22. However, most experience with recurrent river training measures is gained during fairway maintenance. It is essential to study to what extent these recurrent measures can be applied for AFPM. In other words : to what extent is it possible to scale the recurrent measures, applied so far during low flow conditions, up to measures during full bank flow conditions ? Is it sound from hydraulic and morphological points of view ? How can it be realized (design, material, construction) ?

2.3 PRINCIPLES OF AFPM

Active management of the flood plain (AFPM) should be based upon an integrated approach of disciplines such as land use, environment including ecology, socio-economy and hydraulic and morphological behaviour of the river. The subjects of this report concern the hydraulic and morphological aspects.

The principles of AFPM from the hydraulic and morphological point of view consist of controlling the main functions of the flood plain, such as :

- o temporary storage during flood flow condition, thus damping the flood peaks
- o additional discharge during flood flow conditions, thus increasing the total discharge capacity of the river
- o protection of levees both by providing (soil mechanical) stability and by reducing the nearby current velocities and thus the hydraulic load.

The main parameters of the flood plain to be controlled are:

- o the width being the distance between the levee and the outer bank of the outer channel
- o the level
- o the hydraulic resistance or roughness.

The training works for flood plain management can be categorized using these parameters:

- o works affecting the width either by repositioning of levees or by erosion control of the outer bank of the outer channel, in other words: control of outflanking
- o works affecting the flood plain level attempting to control erosion or sedimentation such as plain surface protections and cross-bars
- o works affecting the roughness, related with obstacles in the flood plain such as vegetation and houses.

The most critical part out of this AFPM spectrum is the erosion control of outer banks of outer channels (see Fig. 2.3-1).

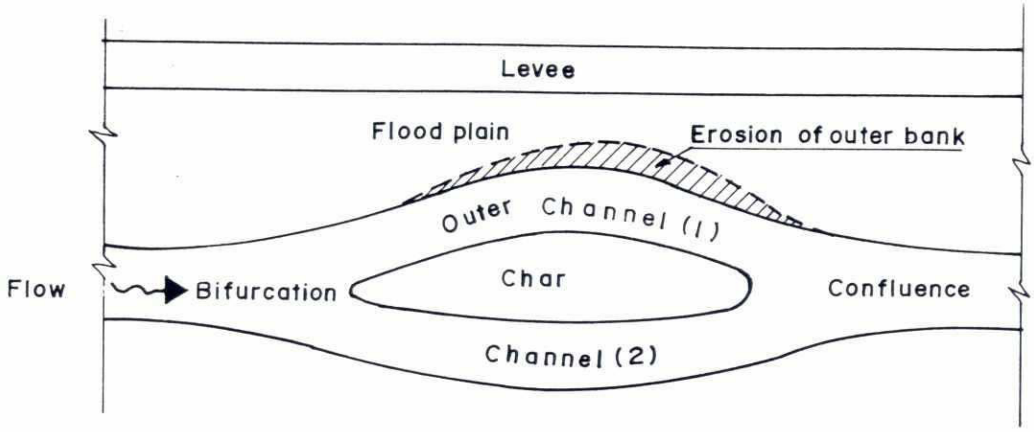

Fig. 2.3-1: Typical planform

As the erosion of the outer bank is cutting down the width of the flood plain towards a critical value, the erosion should either be reduced, or stopped (if the critical width is attained) or reversed into sedimentation (if the critical width is subceeded).

Hence, the timing is essential. Erosion control measures are usually, taken late, often too late, attempting to limit the damage rather than preventing the damage. In this context AFPM means active, timely erosion control to prevent damage of the levees or other (economically) valuable areas. For that purpose a monitoring system is required to detect timely both undesired outflanking processes of existing channels and the development of new channels at critical locations. This makes it possible to close the new channels at relative low cost, before they become main (big) channels. For the control of outflanking various methods exist:

1) measures affecting the distribution of water and/or sediment flow at the bifurcation
2) measures affecting the distribution of water and/or sediment flow over the width of the cross-section of the outer channel
3) measures directly protecting the outer bank

The fundamental difference between the first two methods and the third one is that the first two methods aim at changing the hydraulic and sediment load, while the last method does not change the load, but protects against it.

For all three methods it holds that both permanent and recurrent measures are possible. Proper AFPM means to apply flexible recurrent measures until the planform has been stabilized sufficiently to allow shifting towards more permanent measures.

2.4 PRINCIPLES OF EROSION CONTROL

The three methods of erosion control of the outer bank of the outer channel, as mentioned above are described in the following sections.

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2.4.1 Flow Redistribution at Bifurcations

The principle of this method is that the distribution of the (water and/or sediment) flow at the bifurcation is changed in such a way that the sediment transport into the outer channel (S_1) increases ($+\Delta S$) and/or the water flow into this channel (Q_1) decreases ($-\Delta Q$) as shown in Fig. 2.4-1:

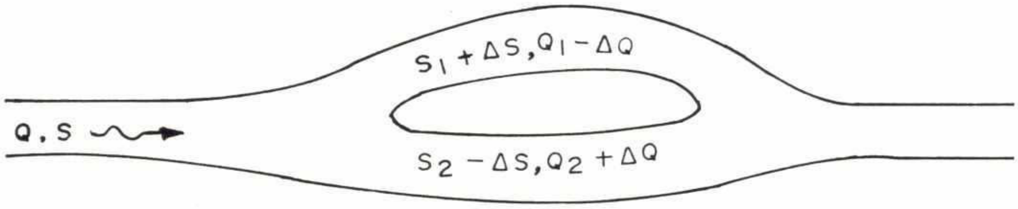

Fig. 2.4-1: Definition sketch

- o **Redistribution of sediment flow**
  An increase of the sediment transport ($+\Delta S$) in the outer channel means exceedance of the natural sediment carrying capacity of the water flow (Q_1) resulting in sedimentation. The sedimentation may affect ΔS but certainly reduces Q_1 thus accelerating the process. The sedimentation will give higher bed levels by which the velocities (the aggressivity) of the outer channel and hence the bank erosion will decrease. The process may be continued until the erosion stops or even until the outer channel is silted up (reduction of braiding index).

  The decrease of the sediment transport in channel 2 means subceedance of the sediment carrying capacity of the water flow (Q_2) resulting in erosion. The river tends to enlarge channel 2, by which the velocities decrease until the carrying capacity fits with the reduced sediment transport. However, as long as siltation is continuing in channel 1, erosion will continue in channel 2.

- o **Redistribution of water flow**
  A decrease of the discharge in channel 1 ($-\Delta Q$) means initially a lower current velocity, and consequently a reduction in the sediment carrying capacity. Sedimentation follows. For small values of ΔQ sedimentation continues until about the same velocities are reached :

$$\frac{Q_1}{B_1 . h_1} = \frac{Q_1 + \Delta Q}{(B_1 + \Delta B_1)(h_1 + \Delta h_1)}$$

in which
B x h = the wet area of the cross-section
B = channel width
h = average water depth

Neglecting the variation of the width (ΔB is zero for small ΔQ values) this leads to

$$\frac{\Delta h_1}{h_1} = \frac{\Delta Q}{Q_1}$$

This means that a new equilibrium is achieved at a shoaling rate equal to the relative reduction of the discharge.

Similarly the scouring rate in channel 2 equals the relative increase of the discharge there :

$$\frac{\Delta h_2}{h_2} = \frac{\Delta Q}{Q_2}$$

This first estimate is valid for discharge variations upto the order of 10% ($\Delta Q/Q$ value). For more substantial variation more accurate computational methods are required.

From these principles it is concluded that the method of redistribution at the confluence is promising enough to be further elaborated. The main questions are :

1) which measures are suitable to realize the desired erosion control ?
2) does such a measure give the desired redistribution of (water and/or sediment) flow at the bifurcation?
3) what are the hydraulic and morphological effects in the channels ?
4) do these effects lead to the final target: at least reduction of the erosion of the outer bank ?

These questions indicate the main components of the required study (see further Section 2.5).

2.4.2 Flow Redistribution in Cross-sections

The principle of this method is that the distribution of the flow in the cross-section of the outer channel is changed in such a way that at the outer bank either the erosion is converted towards sedimentation and/or the velocity is decreased (see Fig. 2.4-2).

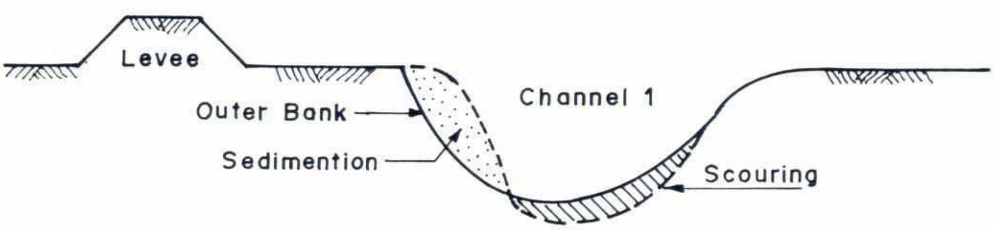

Fig. 2.4-2: Sedimentation at the outer bank

The sedimentation at the outer bank will probably coincide with erosion of the river bed (rather than erosion of the inner bank) thus narrowing the channel.

Possible adaptations of the horizontal velocity distribution are indicated in Fig. 2.4-3.

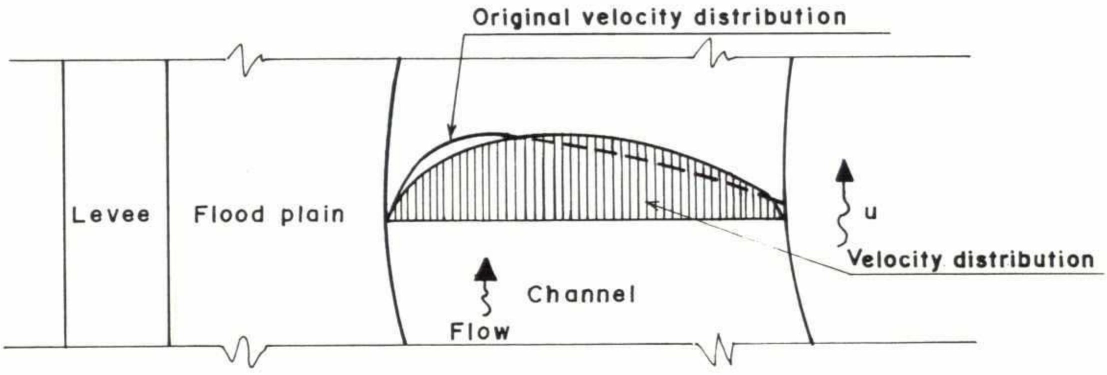

Fig. 2.4-3: Changing flow distribution

Reduction of the velocities near the outer bank coincide probably with an increase elsewhere. Due to the changing velocity distribution the shape of the cross-section will change too. The questions 1, 3 and 4 as raised in the previous section are applicable for this method too : which measures are suitable, and what are the effects in the channel particularly regarding outflanking ?

2.4.3 Protection of Outer Bank

As mentioned before this method of control of outflanking is a defensive one: protecting the outer bank against the impact of flow while not affecting the flow conditions (substantially).

Obviously this method of erosion control is the most direct one (in comparison with the first two methods). However, this does not mean that the method is the most suitable one: probably attractive in terms of effectivity but possibly less suitable in terms of cost-effectivity.

The main question for this method is which measures should be selected.

2.5 SCOPE OF STUDY

Summarizing this chapter it is concluded that the river training within the scope of active flood plain management emphasizes the erosion control in outer channels in order to prevent outflanking or to promote accretion or even closing.

The various methods of erosion control and the main components of study are combined in the following matrix, giving the scope of the required studies.

Component of study / Method of erosion control	Selection of measures	Response of flow at bifurcation	Response of channels	Response of outer bank
1. redistribution at bifurcation	+	+	+	+
2. artificial cutoffs	+	+	+	+
3. redistribution in cross-section	+	-	+	+
4. protection outer bank	+	-	-	-

Matrix : Scope of study

In this report a description of existing measures is presented in Chapter 3 from which a preliminary selection is made (see Part B) indicating the measures to be possibly applied for the various methods of erosion control. In Part C the other components of study (see matrix) are assessed: for method 1 the effects of the measures on the distribution of flow is discussed and for methods 1, 2 and 3 a description is given of the hydraulic and morphological response of the channels to the measures. Finally, in view of the control of outflanking, the relation is described between the changing conditions in the outer channel and the erosion process of the outer bank.

These qualitative assessments of responses of flow at the bifurcation, channels and outer bank should be verified more quantitatively using first order computations including one-dimensional approaches with MIKE 11. These "quantitative" assessments should be part of the research to be done within the FAP 22 approach. Therefore verification results for the qualitative assessments as presented in this "State-of-the-Art" report might-be expected at a later stage of the Project.

3 REVIEW OF EXISTING MEASURES

This chapter describing the existing river training measures comes in three parts. First the (always arbitrary) split is discussed between permanent and recurrent measures. Thereafter both categories have been outlined.

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3.1 PERMANENT OR RECURRENT

As stated before the terminology "permanent and recurrent" indicate the possible lifetime of the measures. Permanent means long term up to say 50 years (engineering time scale), recurrent means short term up to say 5 years. The possible lifetime means that an appropriate design is assumed (strength, stability). Often, the very same measure can be designed either as a permanent or as a recurrent measure depending on among others the choice of construction material and the river conditions.

The hydraulic and morphological effects of a measure can be strong, both initially and as a function of time, so that targets are quickly achieved. In that case it is said that the effectivity of a measure is high. However, an excellent effectivity and sustainability is not enough to call a measure a success. The effectivity and sustainability should be reached in a cost-effective way as elucidated in the following example.

A heavy rip-rap groyne (permanent measure) is constructed to protect the bank of an outflanking main channel. Within a few years the area around the groyne is silted up. The migrating channel is shifting away from the structure and stays away for a long period. From this example the following conclusions can be drawn :

- o the effectivity of the measure is high : immediately the erosion is stopped
- o the sustainability is high : the protection lasts for a long time
- o however, within a few years the groyne is not needed any more and the costly permanent measure lies idle. So the cost-effectivity is low
- o in this example a recurrent measure with a lifetime of a few years should have been a much better solution.

The example indicates that in the migrating channels of big sand bed rivers like the Jamuna there is certainly scope for both permanent and recurrent measures. The recurrent measures are preferred at locations where substantial migrating is to be expected, whilst permanent structures may be preferred at stable locations. As measures tend to reduce migration, gradually more locations will stabilize, enabling to replace recurrent measures by permanent ones. However, even when the main planform of a river becomes entirely stable, recurrent measures may still play an important role as is shown in the following three cases:

1. For a stretch of the Ganga river with one big stable main channel (stable planform at bank full stage) in which various unstable low flow channels are located (unstable planform at low water levels), it was found that river training of the low water bed using permanent measures groynes was far more expensive than training measures using entirely recurrent measures (maintenance dredging and bandalling).
2. In the Rhine River, which is stabilized to a far extent, fairway improvement measures are continuing. Selection of the measures is based upon cost-comparisons between permanent solutions and recurrent solutions (maintenance dredging)

3. On the Lower Rhine agitation dredging (a low cost dredging method) is applied in the lean season to reduce navigational hindrance by shoaling. The top of the dunes is shaven off by propeller action of design vessels (e.g. loaded 6 barge push convoys) accelerating over the shoal.

3.2 PERMANENT MEASURES

In Table 3.2-1 a review of existing permanent river training measures is presented (being a follow up of the preliminary review given in the Inception Report).

Location of measure	Type of measure	Remarks
levee	o man-made levees	for flood control : rationalizing the flood embankment
	o revetments	for protecting levee banks
floodplain	o revetments	against scouring of plains
	o crossbars	-do-
channels	o capital dredging - cut-offs - confinements - closing	for planform corrections
	o revetments	for bottom and bank protection for width limitation and bank protection
	o dikes - permeable groynes - impermeable groynes - bottom cribs, sills - guide bunds	 e.g. needle groynes
	o bottom vanes	in fact sharp crested cribs

Table 3.2-1: Permanent measures

A description of the permanent measures is given in the sections below.

3.2.1 Man-made Levees

The levees or embankments can be natural or man-made. In general the man-made levee consists of an earthen dike often with a road on top (see Fig. 3.2-1). The main purpose of the levee is :

- o to define the high water boundary of the river
- o to reclaim land (back swamps)
- o to protect life and property
- o to provide the basis for an all season road

The crest level of the levee are often around the flood level. The level depends on the degree of safety to be provided for the protected hinterland. Sometimes levels are chosen to be SHWL + clearance in which:

SHWL	=	Standard high water level which is defined in Bangladesh to be the water level which will be exceeded 5% of the time
Clearance	=	About 0.5 m depending on the type of road on top of the levee. Brahmaputa right embankment (BRE) has a clearence of 1.0 m above highest flood level (HFL) considering 1 in 100 years flood frequency.

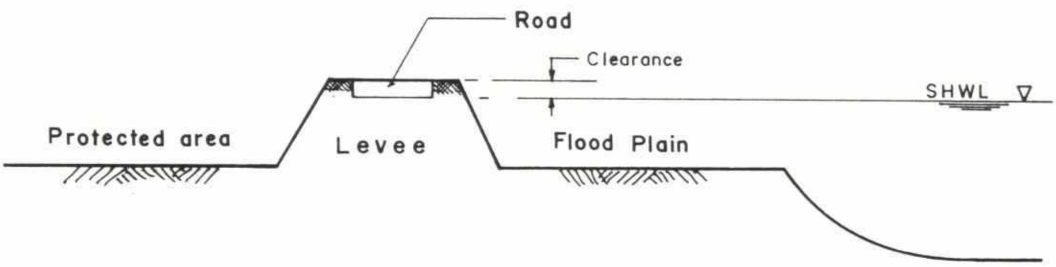

Fig. 3.2-1: Location of levee

Sometimes lower crest levels are chosen to allow the river to spill over the crest. The less protected area is than used as a storage basin for the damping of flood peaks.

The horizontal positioning of the embankment is often historically determined. Mostly the position is determined by the development of land use at both sides of the embankment and the river behaviour. With relative high level flood plain, frequently used, people tend to shift the levee towards the river. Sometimes a river channel erodes the flood plain first and thereafter the levee too; then often a new levee will be constructed farther from the river. Also along the Jamuna the old patterns of previous levees are visible here and there. An important aspect (often getting too less attention) is that the location of the levees determines the stream width and thus the discharge capacity of the river. The height of the levees of a confined river (levees close to the channel) need to be larger than when the levees are located farther from the river. Though the levee is supposed to be a permanent measure the lifetime of levees along the Jamuna is often short, because of aggressive erosion along the outer banks finally undermining the levee.

In case of wide relative high-level flood plains the flow velocities along the levees are below the critical values (above which erosion occurs). There no special protection measures are necessary. Protection of the levee banks may be improved with vegetation or light revetments. Sometimes heavy revetments need to be applied at locations where the flood plain has been completely eroded. Then the revetment of the levee is an extension of the channel revetments. (see Section 3.2.4).

3.2.2 Measures on the Flood Plain

Usually the level of the flood plain is rather stable and the surface gets a natural protection by grasses and other type of vegetation.

Sometimes, however, the velocities over the flood plain are too high during the flood, for instance when the levels of the flood plain are relatively low and scouring occurs, lowering the flood plain levels. For various reasons this scouring may be undesired, for instance in view of the stability of the levee.
To reduce or stop the scouring of the plains, several methods are applied such as :

- revetments
- cross bars
- vegetation

The application of revetments is usually only locally and mostly located near the levee, sometimes combined with the revetment of the levee (see Fig. 3.2-2).

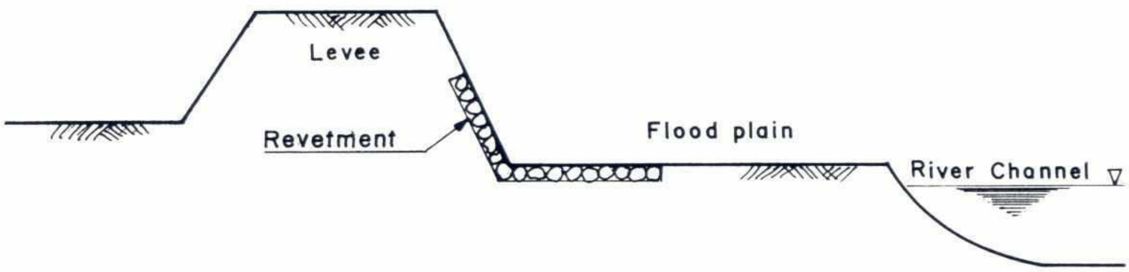

Fig. 3.2-2: Revetment on flood plain

A method to reduce velocities over the full width of the flood plain is to apply crossbars. These are in fact small dikes running perpendicular to the current, from the levee to the river channel.

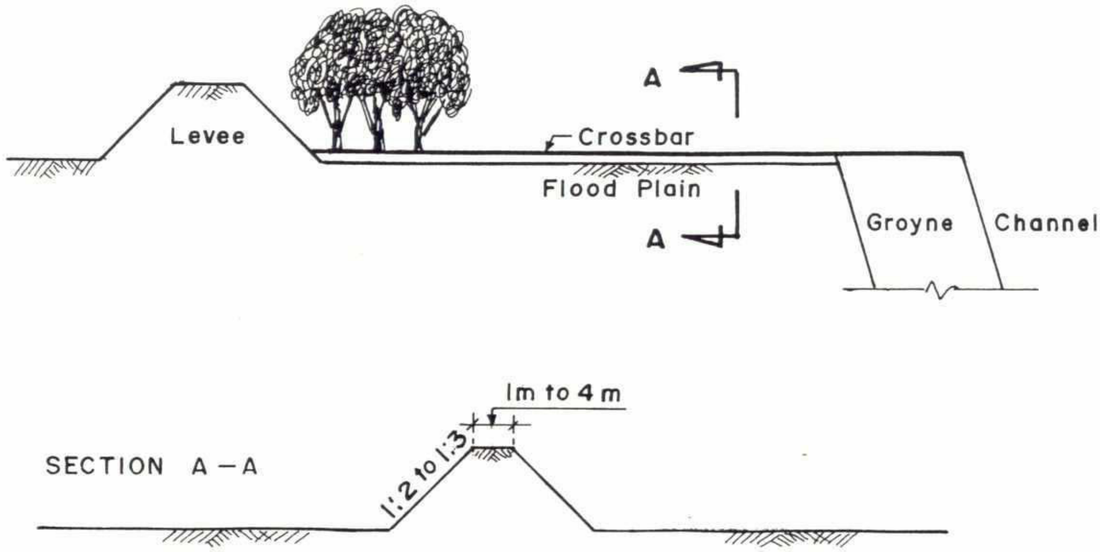

Fig. 3.2-3: Crossbars

Fig. 3.2-3 shows a typical lay-out of a cross-bar as observed along the Jamuna. Cross-bars may be combined with vegetation on a strip along the levee. The heavy variants of crossbars (with crest width of about 4 m) connect groynes with levees.

3.2.3 <u>Capital Dredging</u>

Dredging is one of the measures applied for river training. Capital dredging is utilised for river planform correction by :

(i) cut-offs
(ii) channel closing
(iii) river confinement

(i) Cut-offs
Cut-off can be defined as process by which an alluvial river flowing along curves or bends abandons a particular bend and establishes its main flow along a comparatively straighter and shorter channel. This is a long time consuming process, where the river activity is prominent during high stages. This process can be enhanced by dredging a pilot channel of sufficient dimensions so that the flow, diverted through the channel, will further scour the channel up to the desired dimensions. (For the dimensioning of pilot channels see Pilarczyk 1990, Klaassen and Van Zanten 1989). Due to the specific features of this measure some details related to Jamuna conditions will be described in Section 5.4 as part of the preliminarily selected options for AFPM.

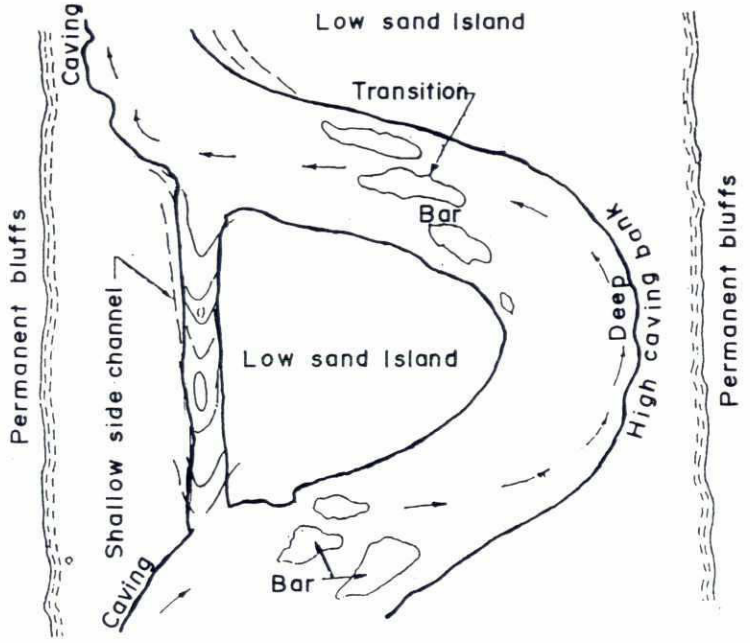

Fig. 3.2-4: Natural Cut-off

(Source: Joglekar, 1971)

Obviously, the desired channel can be fully man-made by dredging also.

(ii) Channel Closing

By dredging in a main channel, the main channel dimension will be increased and with the dredged spoil a secondary channel may be closed. Thus the braiding index of a river is reduced.

(iii) Channel Confinement

In principle dredging works may aim at remodelling the cross-section. This is done in a way keeping the wet area the same, often reducing the width and increasing the depth. This can be schematically shown as follows (see Fig. 3.2-5).

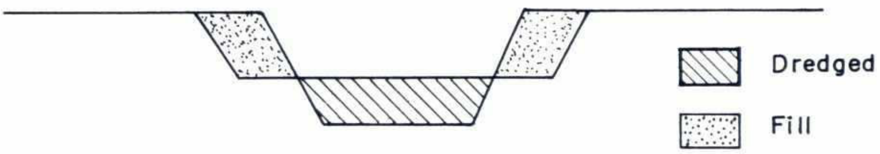

Fig. 3.2-5: River confinement

These types of works are especially done in wide river sections to reduce shoaling.

For these types of permanent measures various kind of dredgers can be used. (Ref. PIANC 1989). The dredger types for river works are :

o bucket dredger
o dipper, front shovel, backhoe, dragline, grab dredgers
o hopper dredger (estuaries only)
o cutter suction dredgers
o dustpan dredger
o bucket wheel dredger

The most common type is the cutter suction dredger.

Capacity of dredgers
To get some insight on the possibilities of dredging as a measure for river training the capacity of dredgers should be compared with the dredging volumes of the measures required.

A few characteristics on the capacity of cutter suction dredgers have been summarized in Table 3.2-2.

IHC Beaver Type	Pump Power (kw)	Diameter Suction Pipe (m)	Maximum Output of Solids per effective hour (m^3/h)
600	390	0.40	550
1200	610	0.45	700
2400	1275	0.60	1150
4600	2550	0.75	1800
8000	3360	0.80	2800

Table 3.2-2: Characteristics of suction dredgers
(Source: IHC Holland)

For the subcontinent effective dredging hours are in the order of 50% if working 24 hours per day in shifts, this means 12 effective hours per day.

Only a part of the maximum output can be realised depending on factors such as :
o dredging depth
o soil characteristics
o pipeline length behind the dredger

An example of the possible output of a Beaver 1200 for 10 m dredging depth the Fig. 3.2-6 is presented.

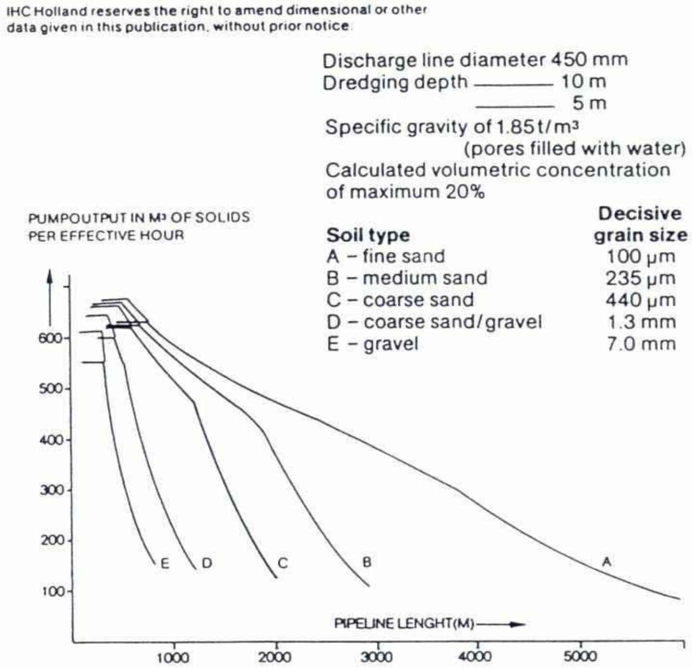

Fig. 3.2-6: Output of a Beaver 1200

(Source: IHC, 1988)

In Bangladesh the maintenance of waterways is undertaken by BIWTA using 8 dredgers with the output as presented in Table 3.2-3.

Type of Dredgers	No of Dredgers	Output in m³ Solids per Dredgers			Totals
		Mean Average Day Production	Monthly	Annual	(m³/year)
D-Class	5	1,810	54,000	648,000	3,240,000
Delta-Class	2	1,520	46,000	552,000	1,104,000
Khanak	1	1,190	36,000	432,000	432,000
Average Annual Output BIWTA Dredger Fleet					4,776,000

Table 3.2-3: Output of BIWTA dredgers

(Source: DHV et al., 1988)

3.2.4 Revetments of Channel Banks

Revetments are used to prevent migration of a channel bank. Often considerable reshaping of the cross-section is required before the revetment can be constructed, (see Fig. 3.2-7).

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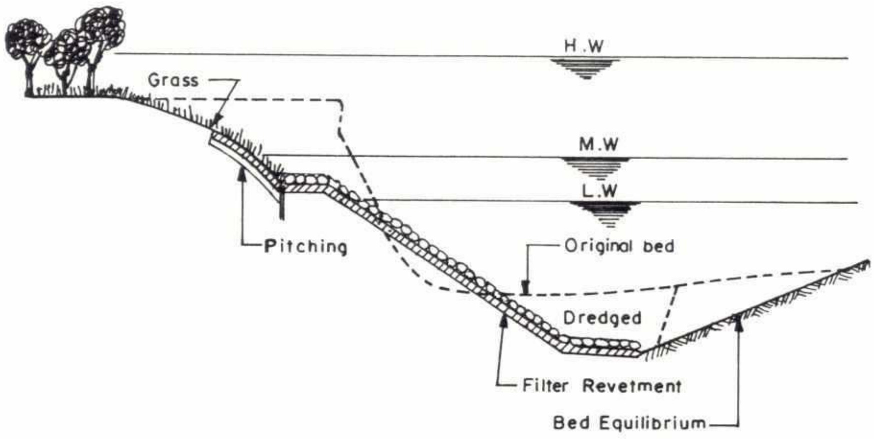

Fig. 3.2-7: Bank revetment

(Source: Jansen et al., 1979)

Revetments are structures built parallel to the current and basically aim at river training at flow conditions all the year around. Obviously the stability should be designed for flood flow conditions. Permanent revetments are constructed of articulated concrete mattress, cement concrete block mattress, woven lumber mattress, willow framed mattress, stone boulder pavement, brick pavement etc. Main purpose of these type of works are to achieve planform stability of river by controlling the bank erosion (Jansen et al, 1979).

When the flow velocity is high the most common types or revetment used in Bangladesh are:

i) Herringbone brick mattresses laid over brick khoa filters. The brick mattresses are enclosed in galvanised wire mesh (see Fig. 3.2-8)

ii) Graded boulder or stone rip-rap revetment over brick khoa or gravel filters

iii) Brick blocks over filter layer

iv) Brick gabions over filter layer

vi) Sand cement block over filter layer

vii) Cement concrete block over filter layer

Below the blocks, previously, 15 cm to 30 cm brick chip filters were used. Nowadays geo-textile layers are being used as filter layer.

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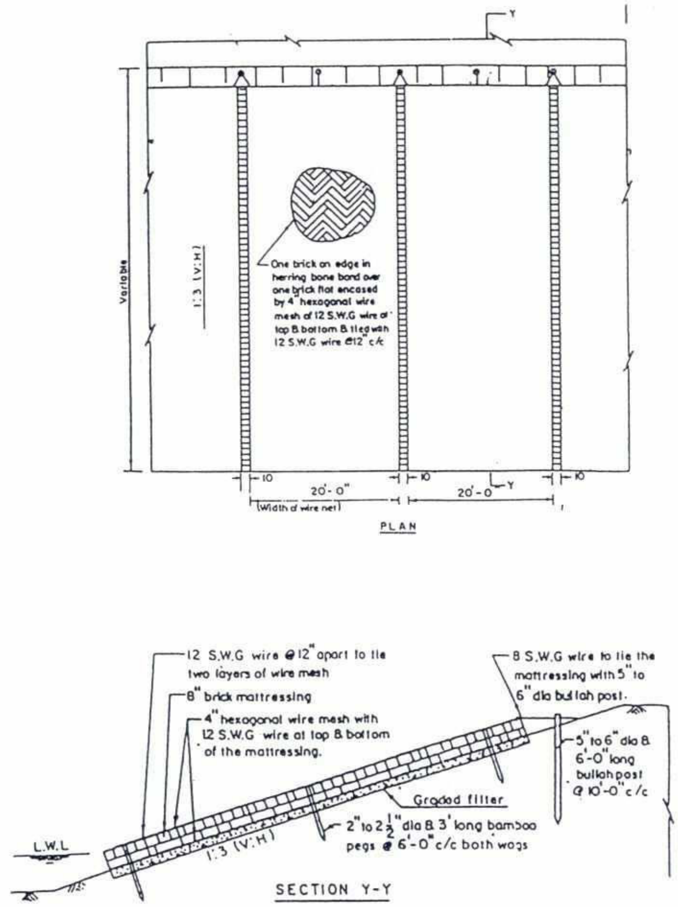

Fig. 3.2-8: Details of brick mattressing

(Source: Huq, 1988)

Articulated concrete slabs over a filter layer have been found effective. Fig. 3.2-9 shows the details of articulated concrete slab.

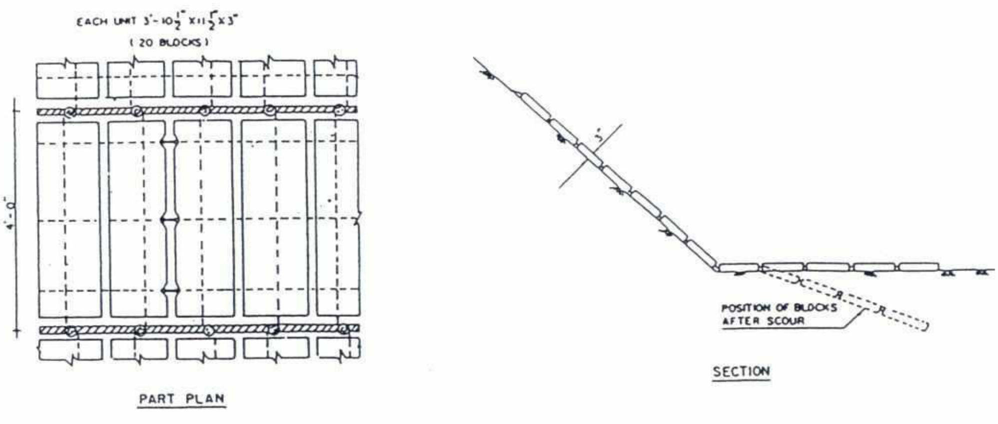

Fig. 3.2-9: Details of articulated concrete slab

(Source: Huq, 1988)

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3.2.5 Dikes in Channels

Various types of dikes may be constructed within a channel. Some, such as groynes, cribs and sills are more or less oriented perpendicular to the current direction, others such as the guide bunds are constructed more or less in the direction of flow. Some notes on these structures are made below .

(i) Permeable Groynes

The permeable groynes consists of one or more rows of piles, or clumps of piles or jacks. (For jacks see Section 3.3.6).

The permanent type of permeable groynes, for instance the needle groyne (see Fig. 3.2-10), consists of steel and/or concrete.

Examples :

- o Steel Needles, as used in Rio Magdalena, Colombia.
  ϕ 0.3 m, spacing about 1 m, water depth 5 m to 10 m, groyne length 150 m.
  Damage reported to be caused by
  - * vandalism
  - * floating debris (water hyacinth)
  - * local scour
- o Concrete pile screens, as used on Java, Indonesia.
  Main Feeder Canals (in canal bends)
  Rectangular shaped 0.4 m * 0.4 m pile, spacing 2 m, water depth up to 10 m, groyne length 30 m, groyne spacing 100 m.

Concrete working bridge on top of screen cum stability beam.

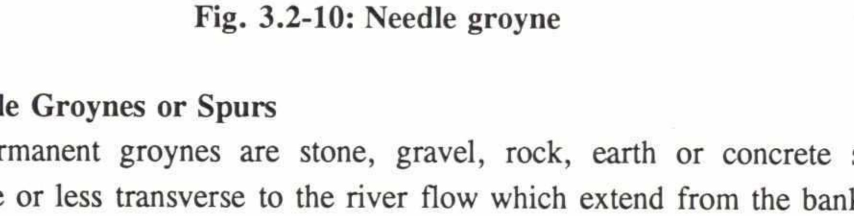

Fig. 3.2-10: Needle groyne

(ii) Impermeable Groynes or Spurs

Impermeable permanent groynes are stone, gravel, rock, earth or concrete structures constructed more or less transverse to the river flow which extend from the bank into the river (see Fig. 3.2-11). They attract or deflect or repel the flow in a channel depending on the shape and angle of the groyne to the river bank. They create a slack flow with the object of silting up the area in the vicinity (see Fig. 3.2-12).

Permanent groynes in Bangladesh are built of earth and armoured with stones, cement concrete blocks, brick blocks or sand cement blocks.

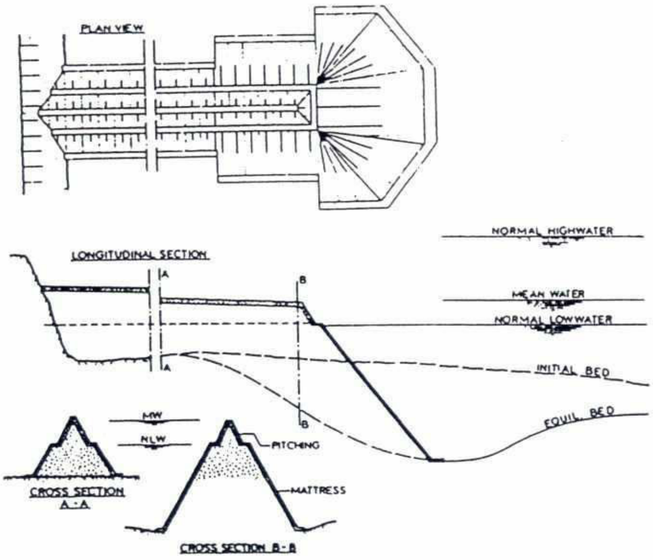

Fig. 3.2-11: Example of a groyne

(Source: Jansen et al., 1979)

Fig. 3.2-12: Flow pattern between groynes

(Source: Jansen et al., 1979)

These type of groynes are permanent in nature and involve huge cost for construction. Sometimes the river width can be stabilised by constructing series of groynes at appropriate places on both banks (see Fig. 3.2-13).

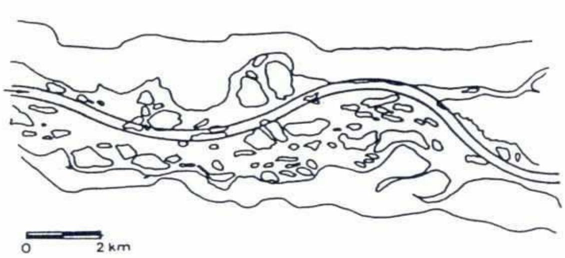

Fig. 3.2-13: Channel regulation on the Rhine downstream of Basle

(Source: Jansen et al., 1979)

Sometimes regulation is realised by a combination of groynes and dredging work (for the channel cut-offs, see e.g. Fig. 3.2-14 and 3.2-15):

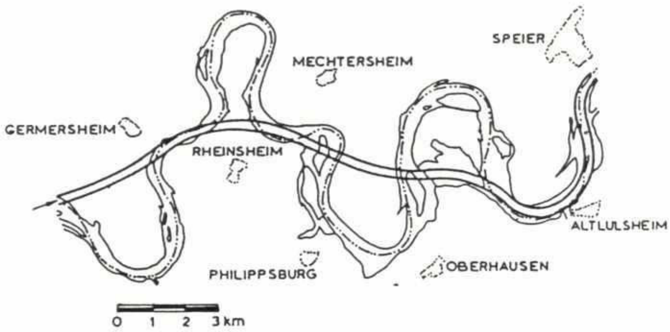

Fig. 3.2-14: Channel regulation on the Rhine upstream of Mannheim (19th Century)

(Source: Jansen et al., 1979)

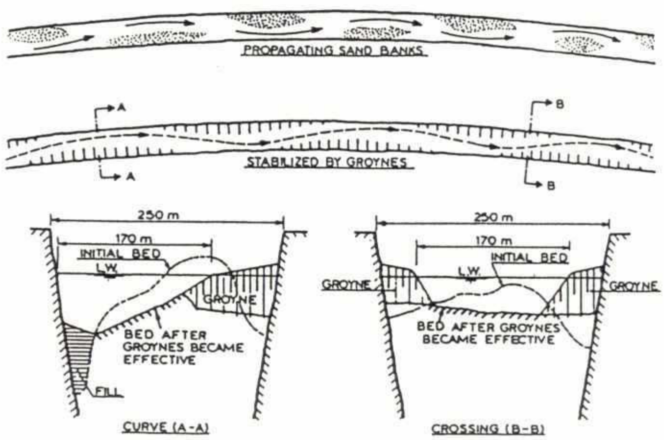

Fig. 3.2-15: Improvement of Rhine rectification by groynes

(Source: Jansen et al., 1979)

(iii) Cribs and Sills

Bottom cribs are considered to be a low type of groynes applied instead of groynes (Huystee, 1987; Struiksma, 1990). As an example reference is made to Fig. 3.2-16.

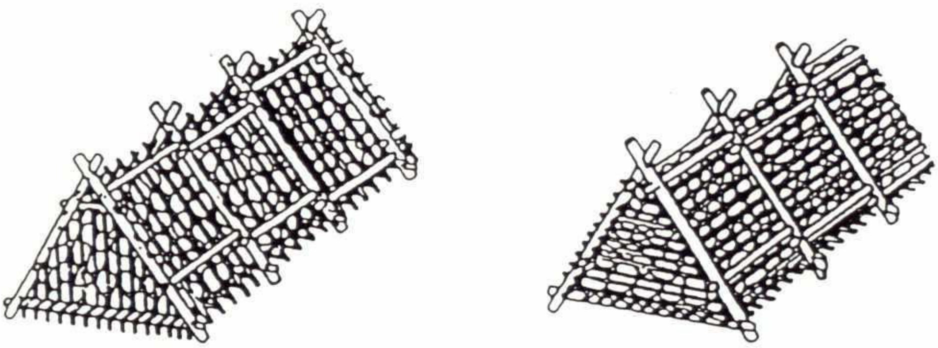

Fig. 3.2-16: Triangular Prism Crib

(Source: UN, ECAFE, 1953)

Sills are in fact low head weirs constructed perpendicular to the current. Series of sills are used to form a cascade. Often sills are applied just downstream of bridges reducing local scour around bridge piers.

The intense transporting power of water at higher stage attacks river banks as well as river bed dangerously. Construction of sills offers a possibility for stopping of degradation. They decrease the sediment transportation capacity of flow by absorbing part of this energy. Sills stabilize the river bottom by controlling the slope of the river flow. Sills create local fix points, between which controlled erosion goes on until a stable slope is attained; after which no further erosion occurs (see Fig. 3.2-17).

River reaches in sharp bends can also be successfully improved by placing submerged sills in the deepest part of the channel, without changing the alignment of the river. Sills improve the sharp bend of narrow rivers by changing the flow pattern and re-distribution of velocities (see Fig. 3.2-18).

Sills are costly structures and these have to be built in series. However introduction of geo-textiles has given an opportunity to reduce some cost of construction by using geo-textile bags. Limited initial erosion between the sills is unavoidable till the bed attains a stable slope. Sills also hinder navigation due to the acceleration of flow over the crest of the sills, according to Pilarczyk, 1990.

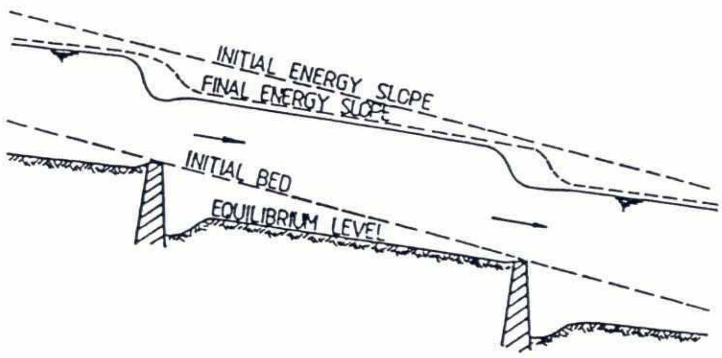

Fig. 3.2-17: Bed fixation by sills

(Source: Jansen et al., 1979)

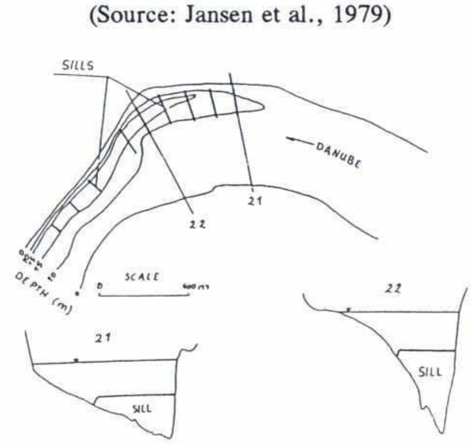

Fig. 3.2-18: Location of submerged sills on sharp bend of the Danube river

(Source: Pilarczyk et al., 1990)

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(iv) Guide Bunds

Guide bunds are used more or less in the flow directions viz. with small angles of attack to direct a flow in a desired direction.

Typical applications of guide bunds are:

1. Near groynes to prevent outflanking of the river behind the groynes
2. Near structures e.g. bridges also to prevent outflanking (see e.g. Fig. 3.2-19).

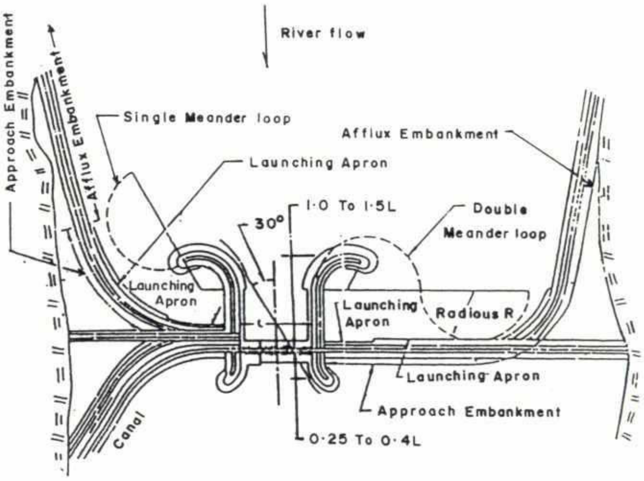


Fig. 3.2-19: Plan of a typical guide bunds

(Source: IS: 8408, 1976)

3.2.6 Bottom Vanes

Mostly bottom vanes are characterized in literature as semi-permanent measures, lasting at least a couple of years. Sometimes a more permanent solution is chosen for instance by constructing the vane as a sharp crested rip rap crib. See for instance Hsieh Wen Shen, bottom vanes in the Lower Mississippi river.

The sharp crest is than the only feature distinguishing the vane from the crib. However, it is an essential one as the sharp crest redirects the overflow more perpendicular to the crest. The orientation of the vanes is therefore fundamentally different from the cribs.

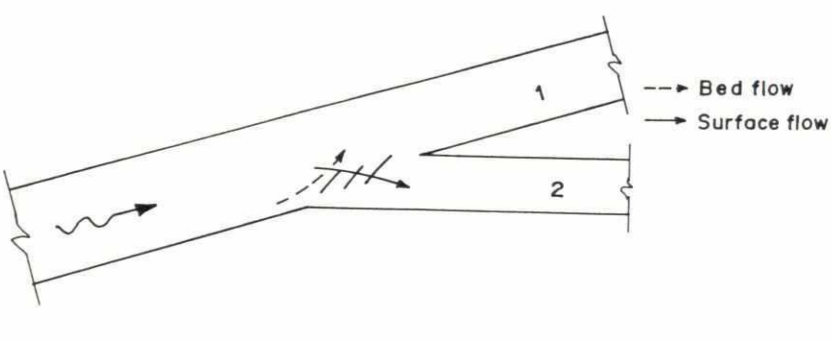


Fig. 3.2-20: Bottom vanes at a channel entrance

Fig. 3.2-20 shows bottom vanes which are located in the entrance to a channel and oriented in such a way that bed flow, and hence bed load, is deflected towards channel 1 and surface flow is deflected towards channel 2. As an example the use of bottom vanes in the Loire River in France (see Fig. 3.2-21) is presented.

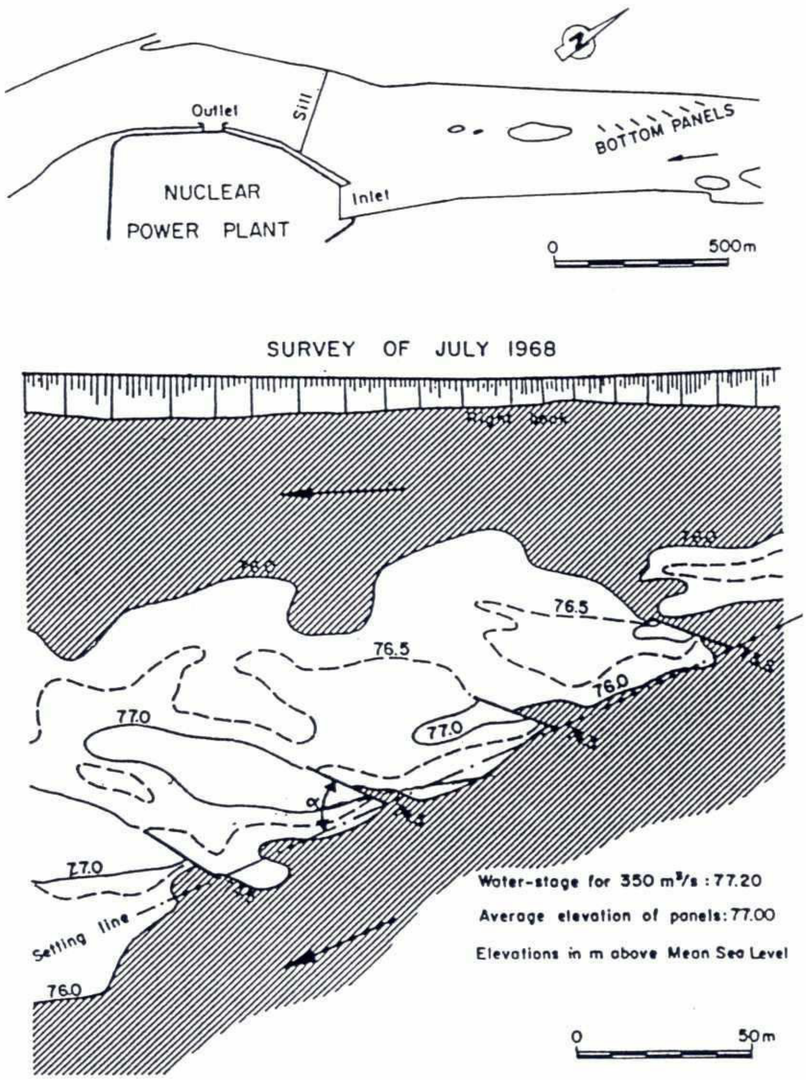


Fig. 3.2-21: Bottom level elevations due to bottom vanes 6 months after construction

(Source: Remillieux, 1972)

The purpose of the bottom vanes (see Fig. 3.2-21) was to close the right arm of the Loire River, during the mid-water and low-water seasons, in order to direct the flow towards the water intake on the left bank. The general characteristics of the vanes are: (1) Length of vanes 25 m, (2) spacing between vanes 50 m, (3) angle of vanes with setting line 45° on an average.

The result of these vanes was the formation of a stable shoal with crest elevation according to the vane elevation (about 1 m above the natural bed level). Note, that as a counter-effect due to this measure the construction of five groynes for protecting the left bank opposite the vane system was necessary.

3.3 RECURRENT MEASURES

In Table 3.3-1 a review is given of existing recurrent measures.

Location of measure	Type of measure	Remarks
flood plain	o vegetation o crossbars	affecting roughness temporary cribs
channels, mostly in the low water bed	o maintenance dredging	including low cost dredging techniques
	o revetments	for bank and bottom protection with e.g. bags and roles
	o dikes - impermeable groynes - permeable groynes	 e.g. with sand bags e.g. - open piles - steel cable - timber pile dike
	- cribs & sills guide bunds	
	o vanes - bottom vanes	 such as the Chao Phraya screens
	- intermediate vanes	applied from bottom to surface such as Potapov screens and Iowa screens.
	- surface vanes	like bandals and floating vanes
	o jacks	like - jetties - cows - porcupines

Table 3.3-1: Recurrent measures

A description of these recurrent measures is given in the sections below.

3.3.1 Flood Plain

The recurrent measures on the flood plain have the objective to prevent degradation. The measures consists of vegetation and/or crossbars.

(i) Vegetation (see Fig. 3.3-1)

Aspects to be taken into account are (Pilarczyk et al., State-of-the-Art, 1990):

- the initial low resistance
- the continuous maintenance requirements
- the threat of decreasing discharge capacity due to overgrowth
- the selection of appropriate species and varieties

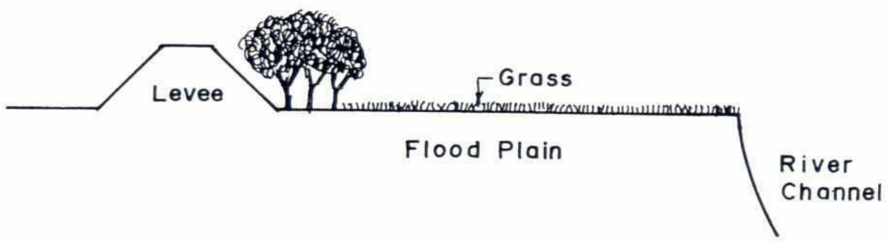


Fig. 3.3-1: Vegetation on the flood plain

The resistance of grass mats on the flood plain decreases as a function of time, see Fig. 3.3-2:

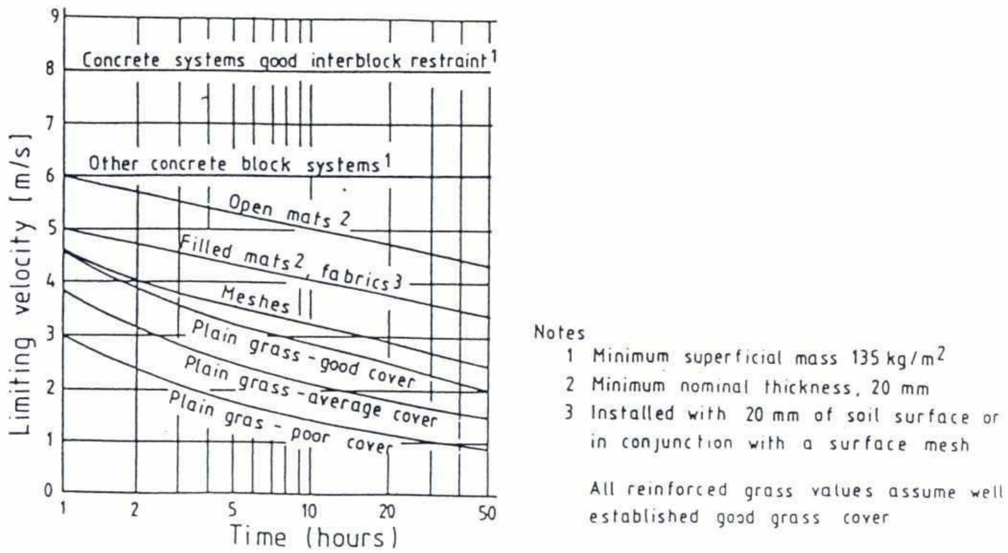


Fig. 3.3-2: Recommended limiting velocities for erosion resistance of plain and reinforced grass against unidirectional flow (after Hewlett et al., 1987)

(Source: Pilarczyk et al., 1990)

For further information on the use of vegetation for river training purpose reference is made to Pilarczyk et al., State-of-the-Art, 1990.

In Bangladesh embankment slopes are protected by turfing. This protects the embankment from raincut and other damage. This measure is also effective against erosion by flood water. When a river spills its banks the flow velocity is reduced considerably.

Protection of river banks by "kash" has been found to be effective. These type of reed grows to a height of 3-4.5 m and a diameter of about 3 cm at the bottom. The reeds are hollow at the bottom and have leaves like sugarcane. The reeds bear white leathery flowers. (UN, ECAFE 1953, p 30)

(ii) Crossbars

Crossbars may consist of local soil covered with vegetation, but other possibilities exist such as :

- o tubes made of polymer grids, woven textiles, wire meshes, etc. filled with sand, gravel, etc.
- o fascine rolls which are brush sausages filled with e.g. stones (see Fig. 3.3-3)
- o bush fence (see Fig. 3.3-4)

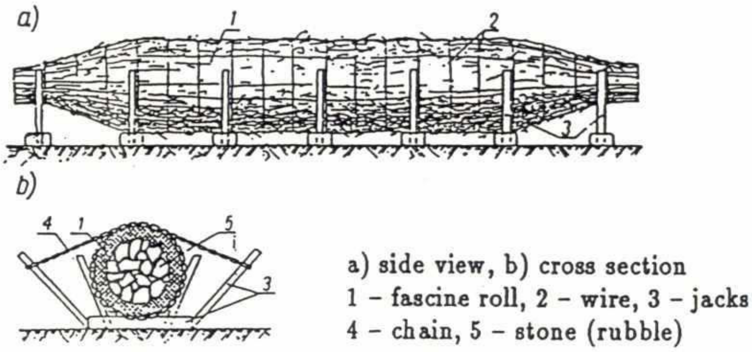


Fig. 3.3-3: Stone-filled fascine roll (after Mamak, 1964)

(Source: Pilarczyk et al., 1990)

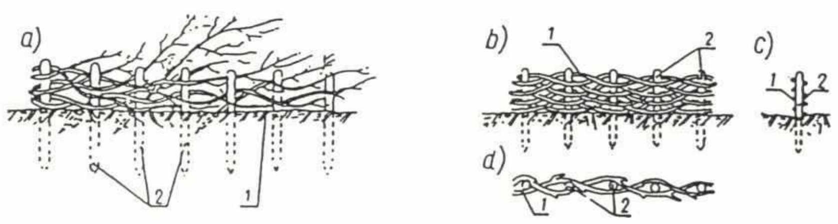


Fig. 3.3-4: Willow fence (after Mamak, 1964), a) during construction, b,c,d) after completion, 1 - willow, 2 - stakes

(Source: Pilarczyk et al., 1990)

3.3.2 Maintenance Dredging

Maintenance dredging is a recurrent measure often applied to maintain navigable channels.

Various types of dredgers may be used for the job (see also Section 3.2.3). The usual types are (Jansen, 1979, p 330) :

- o bucket dredger
- o cutter (suction) dredger
- o dustpan (suction) dredger
- o hopper (suction) dredger
- o bucket wheel dredger

The bucket dredger has the disadvantage that the dredged material has to be transported by barges. The method is not much used outside Europe.

The cutter, dustpan and bucket wheel dredgers are the most general types. As the dustpan is loosening the bed material by jetting there are limitations in the application of dustpans depending on the cohesion-characteristics of the bed material. The popularity of bucket wheel dredger is increasing.

The hopper dredger is only working in the estuaries as the increasing draft during dredging makes the dredger unfit for working in the shallow river areas more upstream.

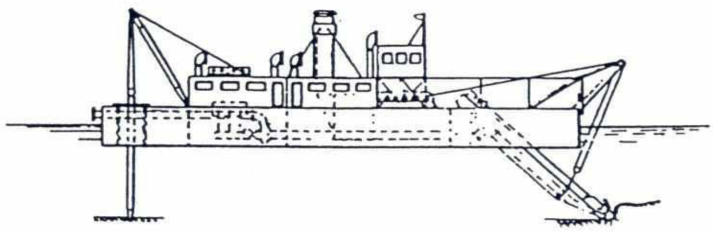


Fig. 3.3-5: Cutter suction dredger

(Source: Jansen et al., 1979)

So the cutter suction dredger (see Fig. 3.3-5) has the widest range of applications. The transport of the water/sand mixture pumped by the dredger is either dumped in barges, or transported via floating and/or shore-based pipelines, or jetted, so-called boom dredging. In some cases the suction dredger is used as an agitation dredger. (see also Section 3.2.3).

Besides these type of dredgers there are other types developed to remove bed material at much lower unit rates, the so-called low cost dredgers. The dredging method is in principle low cost because only money is invested in loosening the bed material. The material is not pumped up and further transported. After loosening, the currents of the river and the density differences due to the sediment concentrations are the causes of the transport (agitation).

Type of low cost dredgers are for instance (Van Oostrum 1989)

- o bed-leveller
- o mud wheeler
- o water injection dredger
- o ship propeller

See the following figures (Fig. 3.3-6 ... 3.3-9):

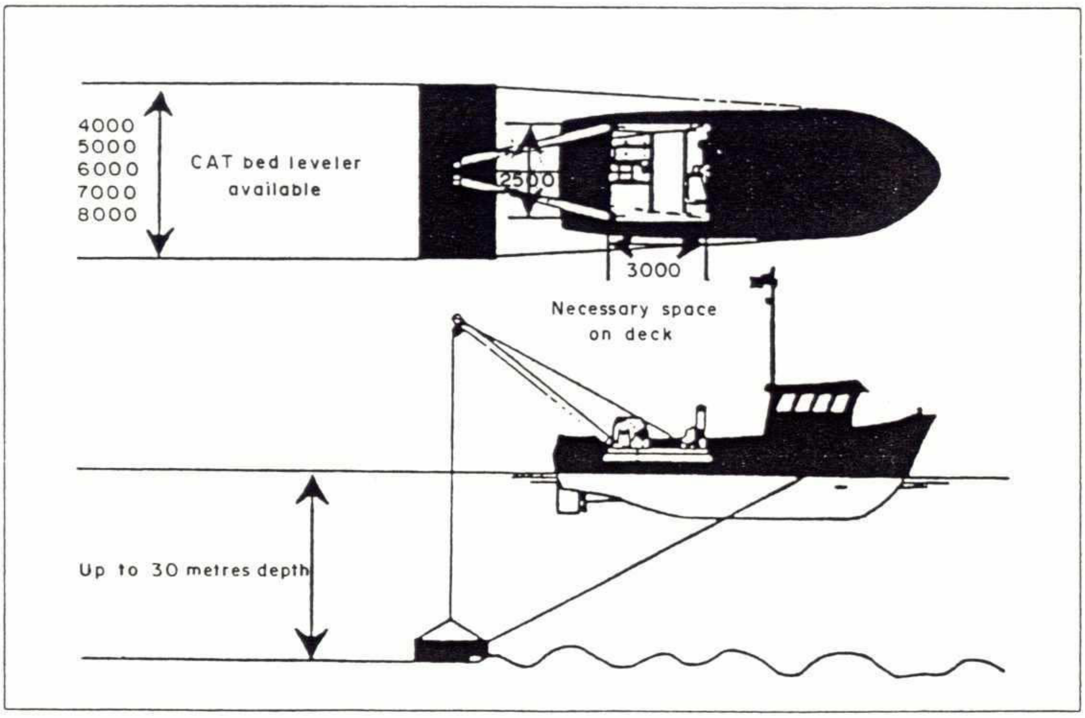


Fig. 3.3-6: Leveller towed behind a tug

(Source: UN, ESCAP, 1989)

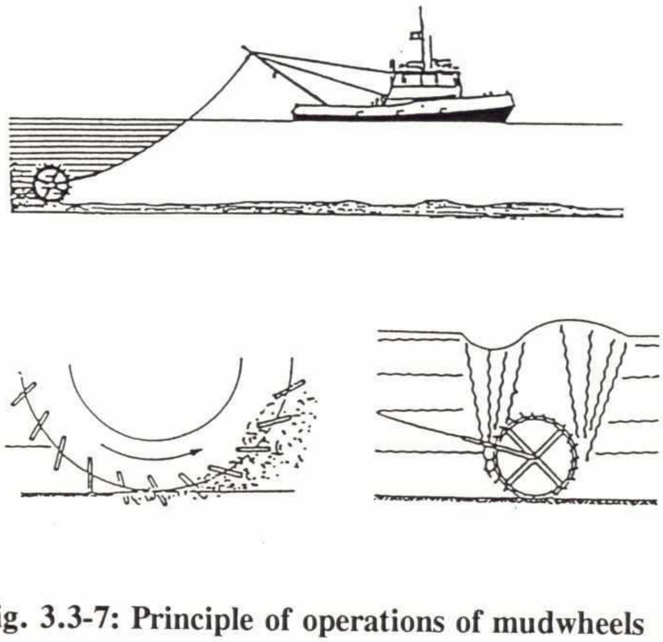


Fig. 3.3-7: Principle of operations of mudwheels

(Source: UN, ESCAP, 1989)

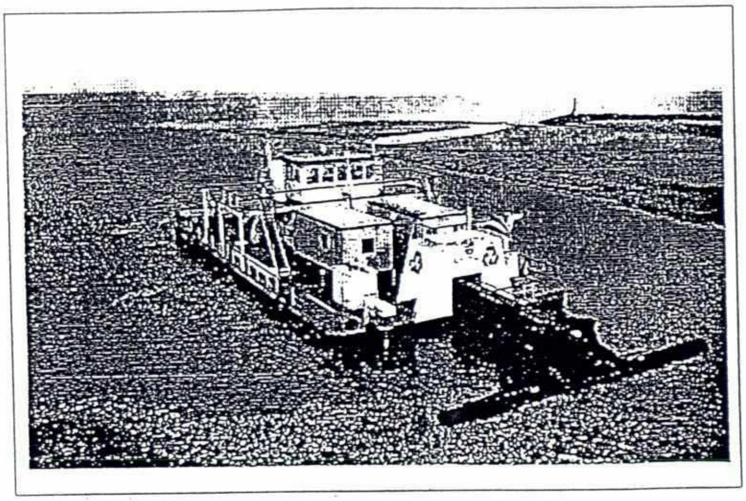


Fig. 3.3-8: The "Jetsed", (Courtesy : Volker Stevin)

(Source: UN, ESCAP, 1989)

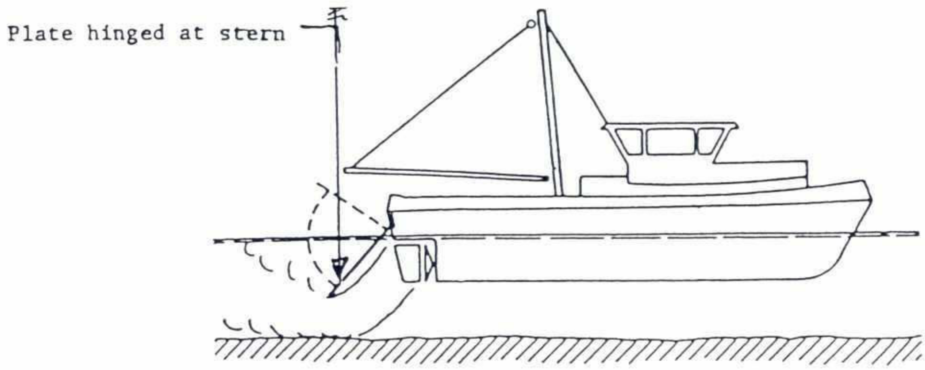


Fig. 3.3-9: Agitation by using ships propellers

(Source: UN, ESCAP, 1989)

Obviously the last method is only effective in shallow water.

As low cost methods are all based on agitation the effectivity is improving in case of :

- o sufficient current velocity
- o a suitable longitudinal river profile
- o characteristics of the bed-material (grain size, cohesion)

The longitudinal river profile should have the following typical shape (see Fig. 3.3-10):

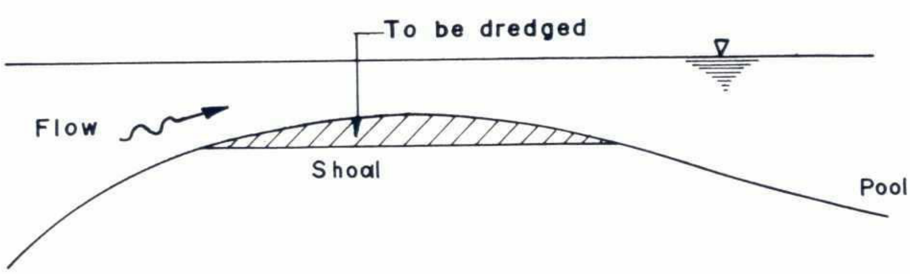


Fig. 3.3-10: Typical longitudinal profile

So the length of the shoal and the nearby presence of a suitable settling area are important factors affecting the effectivity. Usually this type of dredging is applied for the removal of limited layers of bed material (surface dredging).

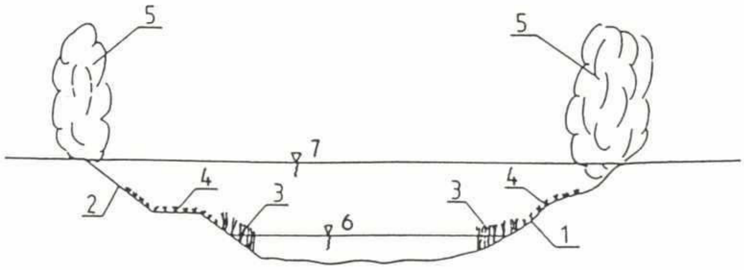
3.3.3 Revetments

Recurrent type of revetment are often employed in case of fund constraints. The light structures are effective where the flow velocity is not too high.

The structures may consist of bamboo or wooden pallasiding. Sometimes gunny bags filled with soil, fascine mattresses, etc. are placed. In China alternate layers of sorghum stalk and clay are used for bank revetment work. Gunny bags are also used to fill deep river scouring holes in the river bed on emergency basis for protecting against river bank failure. In India bank revetment with gunny bags filled with earth has been used.

Two examples of 'low cost' revetments are given here :

- o vegetation (see Fig. 3.3-11)
- o bags or tubes (see Fig. 3.3-12)



1 - old bank	2 - new composite profile	3 - reed	4 - grass
5 - trees and shrubs	6 - mean water level	7 - high water level	

Fig. 3.3-11: Vegetation zones on a river bank

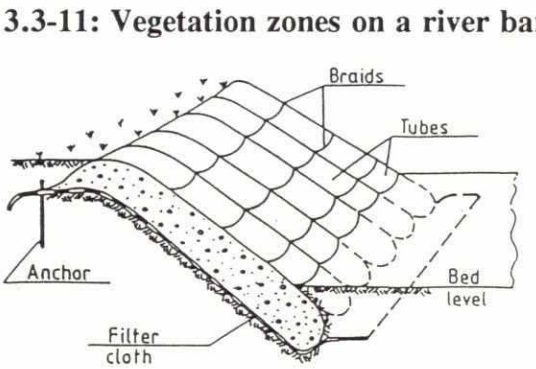


Fig. 3.3-12: Use of tubular gabions in bank protection

(Source: Pilarczyk et al., 1990)

3.3.4 Dikes

Recurrent type of dikes are considered here :

- o impermeable groynes
- o permeable groynes
- o cribs and sills
- o guide bunds

(i) Impermeable Groynes

Recurrent impermeable groynes may consist of sand bags or tubes (e.g. nylon tubes filled with sands). Also a double row of fences filled with sand and/or stones are used.

(ii) Permeable Groynes

Permeable groynes of a recurrent type are mostly applied for river bank protection at locations where the conditions are moderate.

Various types of open groynes exist such as:

- o open needle groyne including timber pile dikes
- o steel cable groyne
- o tree groynes (see Joglekar, 1971).

Some examples of such groynes are presented below.

Needle groynes (see Fig. 3.3-13) are made of concrete, steel, bamboo or timber. The latter is also called timber pile dike.

Examples of steel pipe needles are found e.g. in the Rio Magdalena, Colombia; Concrete needles are found in e.g. Indonesia.

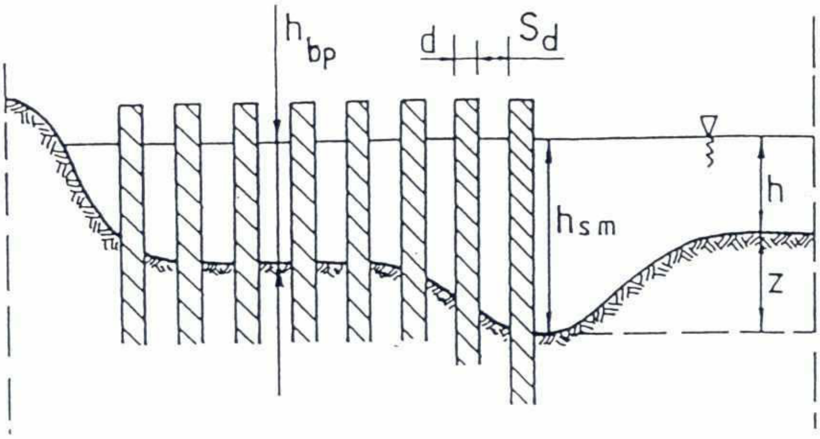


Fig. 3.3-13: Descriptive sketch of a permeable (needle) groyne

(Source: Pilarczyk et al., 1990)

Examples of bamboo needle groynes are found in the Irawaddy River in Myanmar (Prins, 1990). For examples of timber pile dikes see for instance Chang 1988. Same examples of timber pile structures are also presented in Fig. 3.3-14.

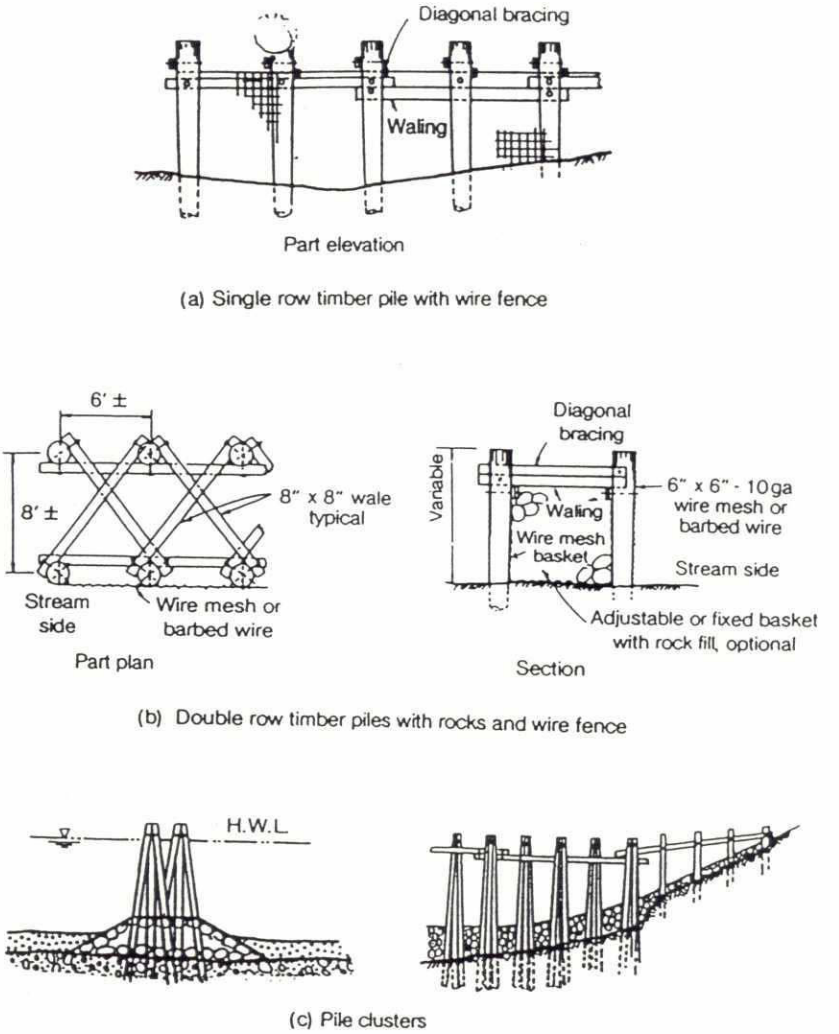


Fig. 3.3-14: Various arrangements of pile groynes

(Source: Chang, 1988)

For further descriptions of pile dikes see Petersen 1986 and Pilarczyk et al., 1990.

Examples of steel cable groynes are found in the Niger River (Prins 1990). Basically these are pile groynes with a wide spacing with in between cables or a net to catch floating debris and thus increasing the hydraulic resistance and slackening the flow behind the groyne.

In Bangladesh permeable groynes are usually made of bamboo or wooden bulla (type of tree). However, the water depth and the scouring depth in the main channels of for instance the Jamuna or lower Meghna were too high for these materials. Hance the Project is investigating the possibility of making use of the favourable effects of permeable groynes but to avoid the disadvantages mentioned. A typical example of a timber spur in a smaller river (Gumti river) is given in Fig. 3.3-15.

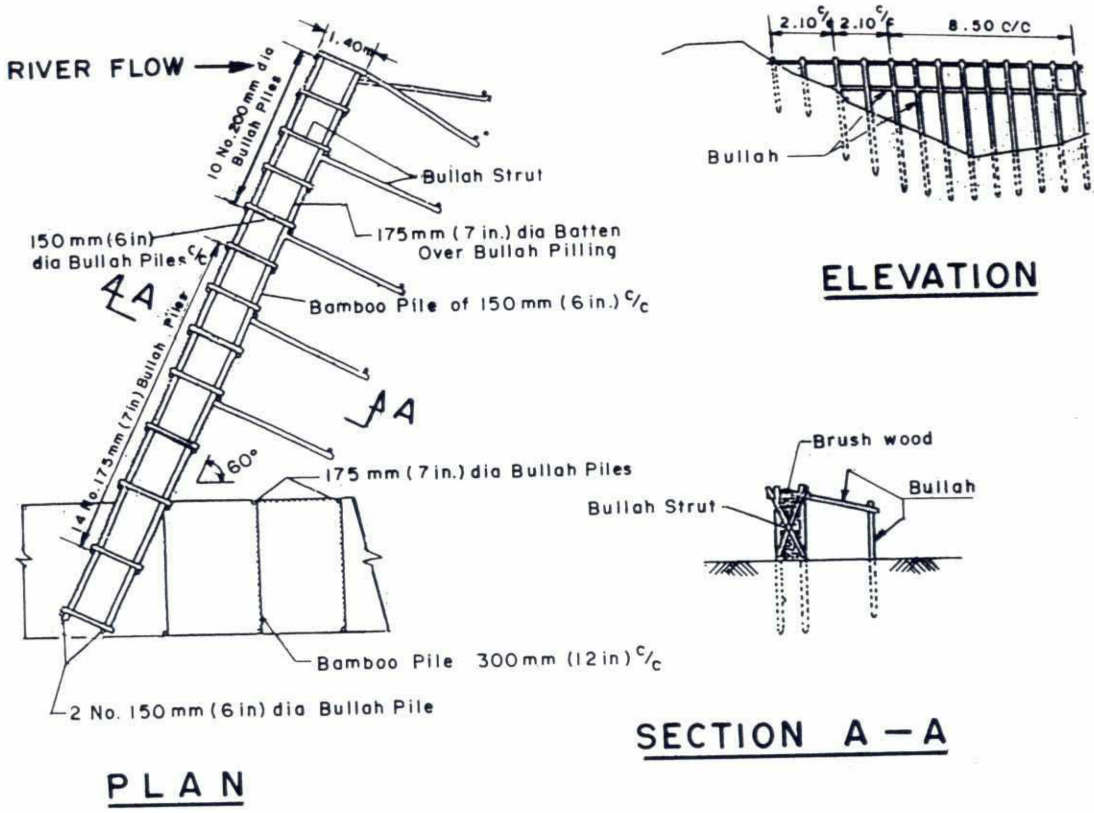


Fig. 3.3-15: Typical timber spur installed along Gumti river

(Source: Working Paper No.12, FCD III, BWDB)

(iii) Cribs and Sills

The recurrent types of cribs and sills are constructed in the same way as the impermeable groynes :

- o bags or tubes filled with sand
- o double fences filled with sand or other material, sometimes brush
- o a special category are inflatable sills.

(iv) Guide Bunds

Guide bunds are mostly applied near bridges, dams and such other structures. In this respect guide bunds serve a two-fold purpose. Firstly, they protect the approach embankments of the bridge from attack and secondly, they control the river and induce it to flow more or less axially through the bridge. When a river is bridged from bank to bank, guide bunds are obviously unnecessary, but with constriction, they become indispensable. It might be clear that guide bunds, due to their objectives, are less suitable as recurrent measure.

3.3.5 Vanes

Vanes consist of three types, depending on their vertical position above the river bed :

- o bottom vanes
- o intermediate vanes
- o surface vanes

For some publications on vanes, see Code V in the list of references.

(i) Bottom Vanes

Bottom vanes are installed on the river bed to deflect the lower part of the flow. The other objective is to change the direction of the bed load. Typical examples of applications of bottom vanes are :

1) at a bifurcation (see Fig. 3.3-16)

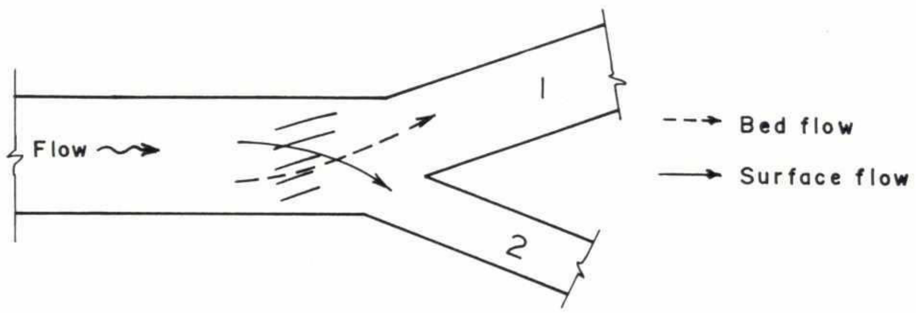


Fig. 3.3-16: Bottom vanes at a bifurcation

The objective here is to direct the bed load transport to channel 1 to promote sedimentation there.

2) at a port entrance (see Fig. 3.3-17)

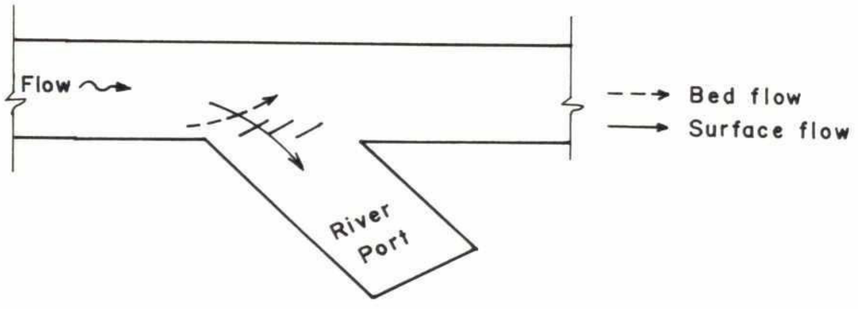


Fig. 3.3-17: Bottom vanes at port entrance

The objective here is to reduce maintenance dredging in the river port.

3) in a river bend (see Fig. 3.3-18)

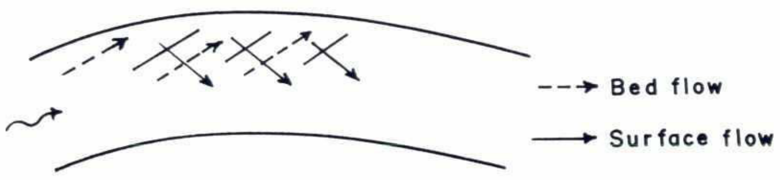


Fig. 3.3-18: Bottom vanes in river bend

The objective here is that the bottom vane tends to generate a special flow which suppresses the spiral bend flow thus reducing scouring depths along the outer bank. (Odgaard and Spoljaric, 1989)

4) in a river crossing sometimes bottom vanes are placed in a river crossing as shown in Fig. 3.3-19.

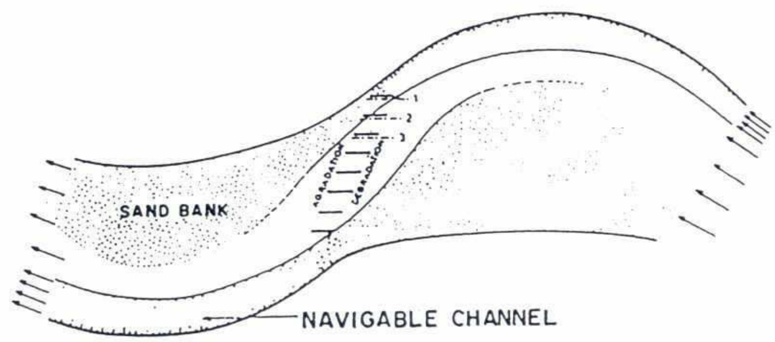


Fig. 3.3-19: Lay-out of bottom vanes across the river

(Source: Jansen et al., 1979)

The principle and functioning of bottom vanes can be seen from Fig. 3.3-20.

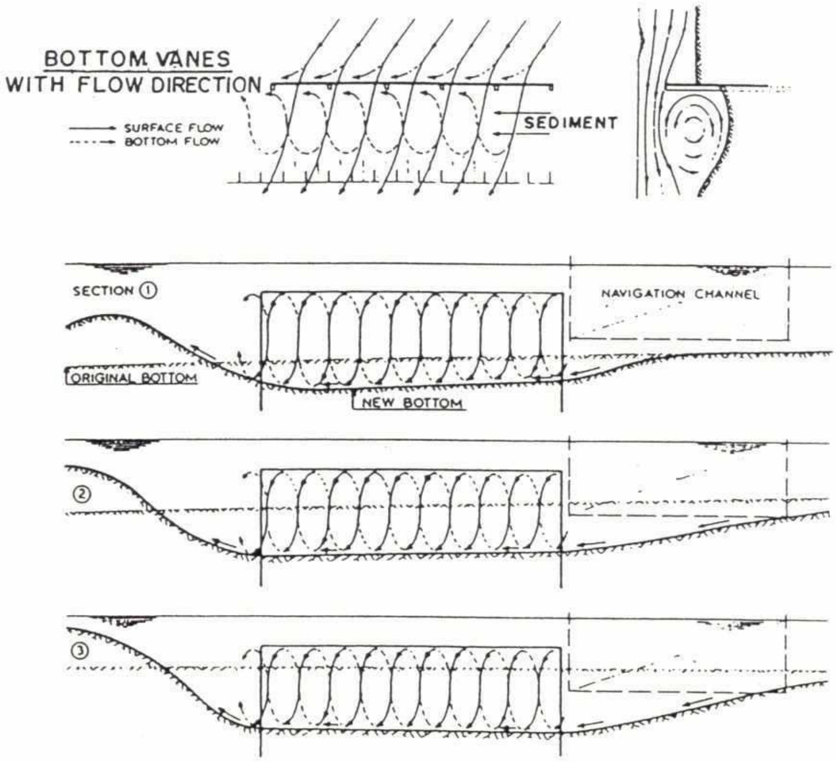


Fig. 3.3-20: Principles of bottom vanes in a river crossing

(Source: Jansen et al., 1979)

5) along a river bank

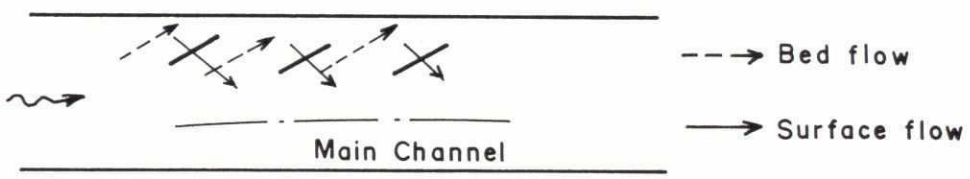


Fig. 3.3-21: Bottom vanes along river bank

Here (see Fig. 3.3-21) the purpose is to promote sedimentation near the left bank and deepen the main channel. Obviously also vane rows along both banks are possible.

An example of the last application is the screens in the Chao Phaya, Thailand. The bottom vanes are placed their to narrow the navigable channel to improve the least available depth. The vanes are made of vertical needles of steel with in between concrete slabs, see Fig. 3.3-22.

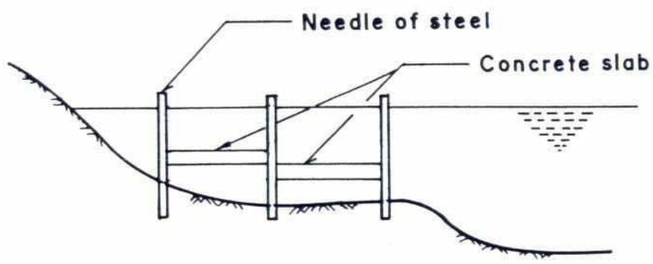


Fig. 3.3-22: Principle of Chao Phaya screens

At both ends of the slab steel rings are fitted enabling to shift the slab vertically along the adjacent needles. Thus a flexible structure is obtained. In case of settling or scouring additional slabs can be added to raise the vane crest.

(ii) Intermediate Vanes

Intermediate vanes are positioned between river bed and water surface. The principles are extensively described by Potapov (1950). Examples of submerged vanes are given in Fig. 3.3-23.

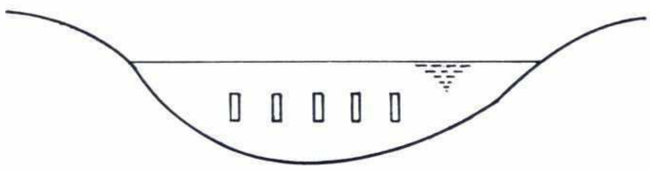


Fig. 3.3-23: Intermediate vanes in river

(Source: Potapov, 1950)

In small deep channels these vanes generate two spiral flows, one in the upper part of the cross-section and one in the lower part with a reversed direction of rotation.

Another type of intermediate vanes consists of rows of smaller foils. These vanes are sometimes called Iowa vanes. Various applications for the lay-out of Iowa vanes, are suggested by Odgaard and Wang (1991).

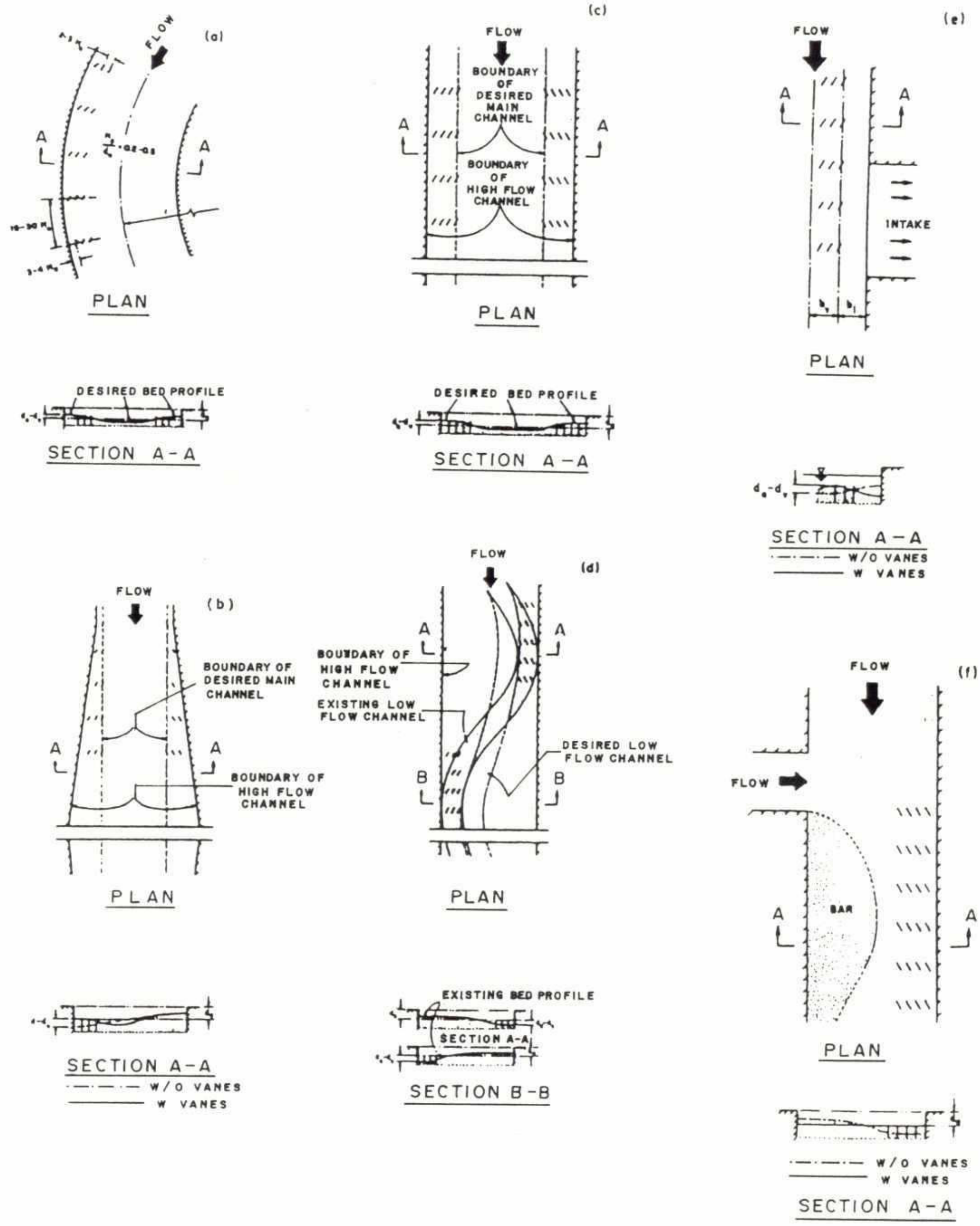


Fig. 3.3-24: Typical vane layouts for stream-bank protection and/or shoaling control: (a) in curved channel; (b) in widened bridge waterway; (c) in navigation channel or bridge waterway; (d) in channel with alternate bars or meanders; (e) at water intake; and (f) at river confluence

(Source: Odgaard & Wang, 1991)

(iii) Surface Vanes

Surface vanes consist of :

1) floating panels, anchored or moving behind barges
2) fixed vanes mounted on a frame such as bandals.

An usual distinction also in wordings is that the panels have much smaller width/draft ratios than the bandals.

o floating panels

The surface panels placed obliquely in the current, cause a deviation of surface flow and due to the acceleration under the panel, also a deviation of the bottom flow in the other direction (see Fig. 3.3-25). The result is a helical flow downstream of the panel. The increased sediment transport follows the bottom flow. The principle has been applied for the protection of water intakes against sedimentation, and the reduction of erosion at outer bends (Batalin, 1961).

The principle mentioned above has also been applied by French engineers for scouring a channel (Rousselot and Chabert, 1961). They grouped a number of surface panels together in a so-called scouring barge (see Fig. 3.3-26). The result was erosion and sideways movement of the sediment over a certain distance downstream of the barge. If this distance is too small the barge can be shifted further downstream after a number of days.

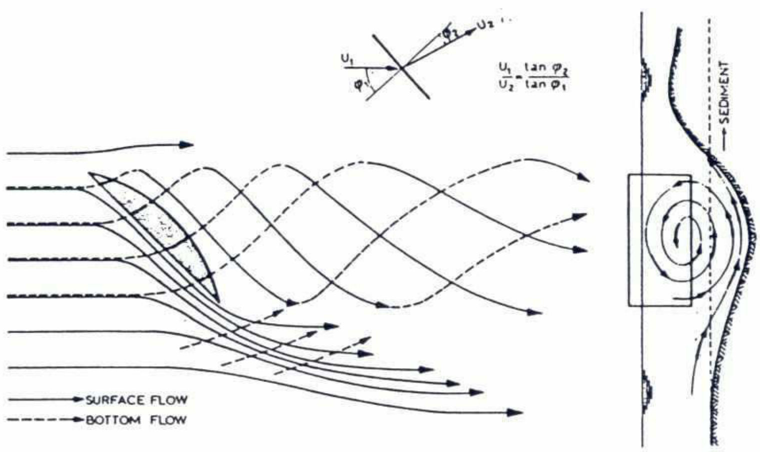


Fig. 3.3-25: The principle of a surface panel

(Source: Jansen et al., 1979)

Applications of floating vanes in China are described by Wan Zhaohui, 1992. Mostly a set of vanes was used to prevent sedimentation of an irrigation intake (see Fig. 3.3-26).

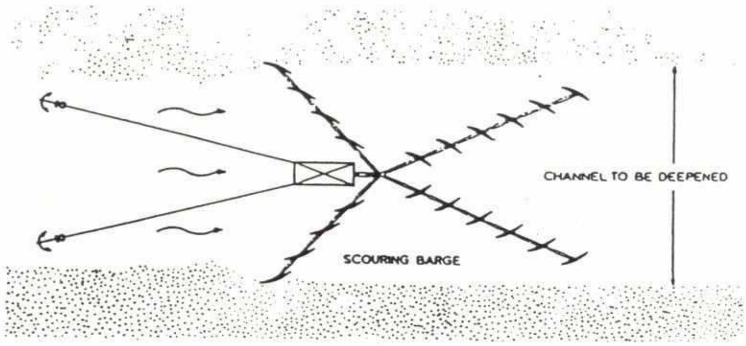


Fig. 3.3-26: Surface panels united in a 'scouring barge'

(Source: Jansen et al., 1979)

The vanes are mounted under a frame. The whole system is floating and anchored in the river. From the experience it was found that the system is an effective measure in releasing sedimentation and diverting the flow. However, due to intensive maintenance efforts, the system was dismantled after a number of years. A more extensive description will be presented at a later stage of the project.

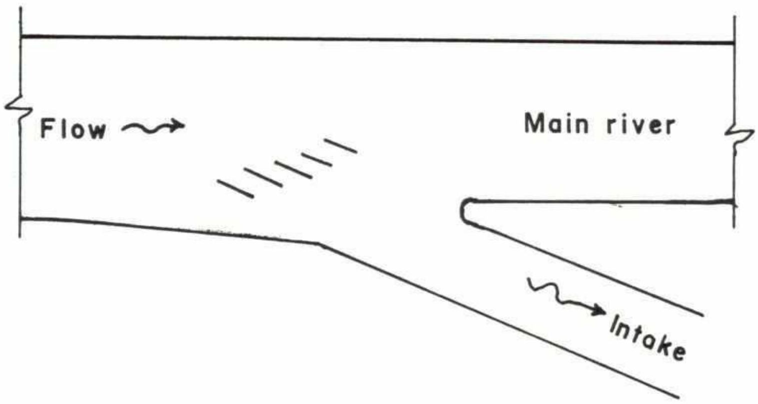


Fig. 3.3-27: Floating vanes before intake

o Fixed surface vanes

Fixed surface vanes like bandals are applied for various purposes (see Fig. 3.3-28). Two important objectives are:

- o fairway improvement
- o closing of secondary channels

For the principles of bandalling, see Jansen, 1979. For experiences with bandalling on the Brahmaputra and the Ganges reference is made to Gogoi, 1982 and Wilkens, 1988. respectively.

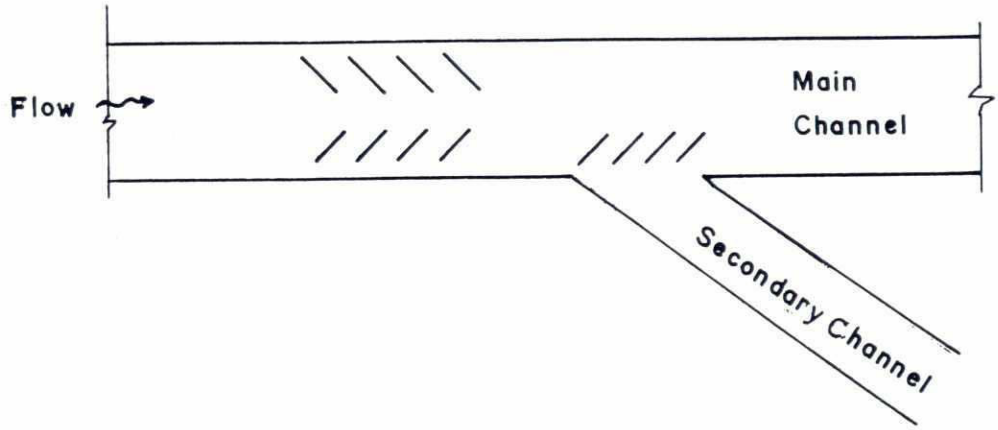


Fig. 3.3-28: Application of bandals

A "bandal" consists of a frame-work with bamboos driven into the river bed and set 0.6 m apart with horizontal ties and supported by struts placed at every 1.2 m. To this bamboo frame-work, "bamboo mats", are tied with coir rope to the horizontal tie at the water level. The bamboos used on the framework are usually 3 to 6 m in length and the "mats" are made of bamboo (0.75 - 1.0 m) wide strengthened at the edges by strips of split bamboo.

"Bandals" are placed at an angle of 30 to 40 degrees, inclined downstream to direct an additional volume of water into the desired channel and to increase the velocity in it. They do not stop the flow of water but merely check it. This causes that part of the sediment transport is carried outwards under the "mats" and deposited in ridges parallel to and behind the "bandals". Thus a channel confined between "bandals" is formed, with sand-banks on either side, and thus the whole discharge of the river is directed through this channel. The plan and elevations of bandals or schematically presented in Fig. 3.3-29.

In reality the construction of the bandals and the process of erosion is somewhat more complicated. In order to prevent deviation of the sediment transport from the sand banks towards the channel a small opening is left between mat and sand bank. As a consequence the surface flow deviates towards the channel to be eroded, while the bottom flow together with the bed load passes under the mat (Mohan and Singh, 1961). Flow acceleration under the mat causes erosion under the bandal and directly downstream of it. In order to maintain the flow acceleration the mats have to be lowered a number of times. The acceleration causes the flow to turn to a direction more perpendicular to the direction of the mats; helical flow results which transports the sediment away from the channel, Thus not only the surface flow but also a part of the bottom flow without sediment, however is brought towards the channel to be eroded.

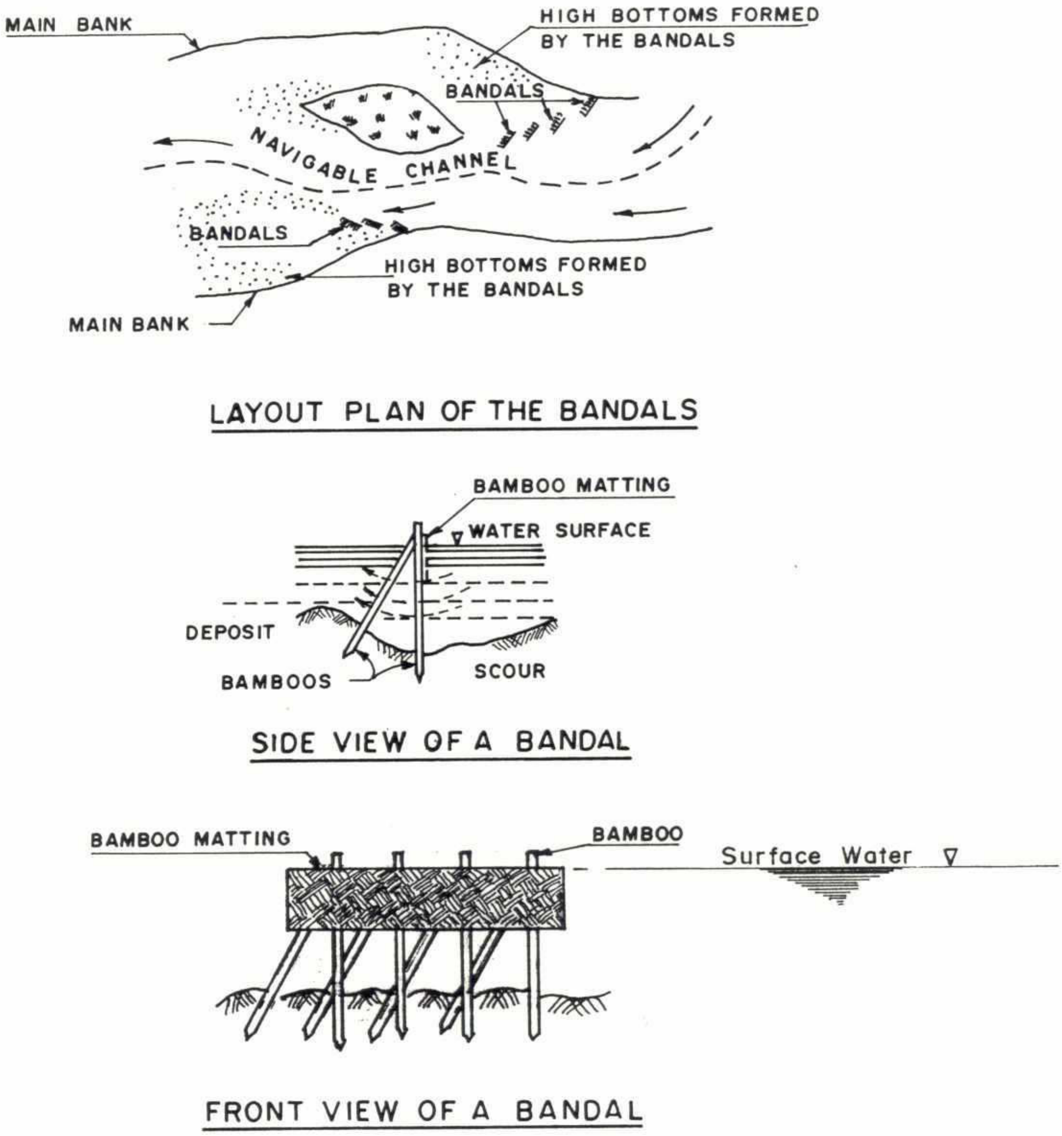


Fig. 3.3-29: Plan and elevation of bandals

(Source: Nishat, A., 1986)

The feasibility of bandalling depends on a number of conditions. A sand bank must be present on at least one side of the channel to be eroded, and preferably on both sides. It must not be too deep when bandalling starts, but also not too shallow, so that a sufficient quantity of water is brought towards the channel. The river must not fall too fast, because sufficient erosion can only be achieved after a number of weeks. The sand must not be too coarse and not too fine.

3.3.6 Jacks

Many type of jacks exist in the world, such as:

- o Jetties
- o Cows
- o Porcupines
- o Retards

Some examples are given below:

(i) Jetties

Examples of a jetty field (see Fig. 3.3-30) and steel jacks (see Fig. 3.3-31) are given by Chang, 1988.

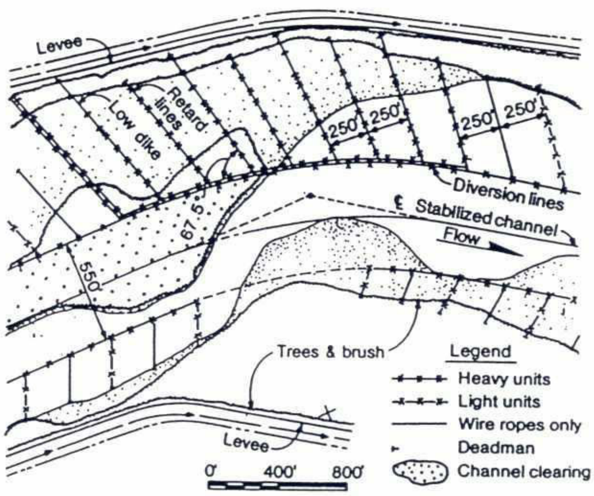


Fig. 3.3-30: Typical jetty field layout - Rio Grande

(Source: US Army Corps of Engineers in Chang, 1988)

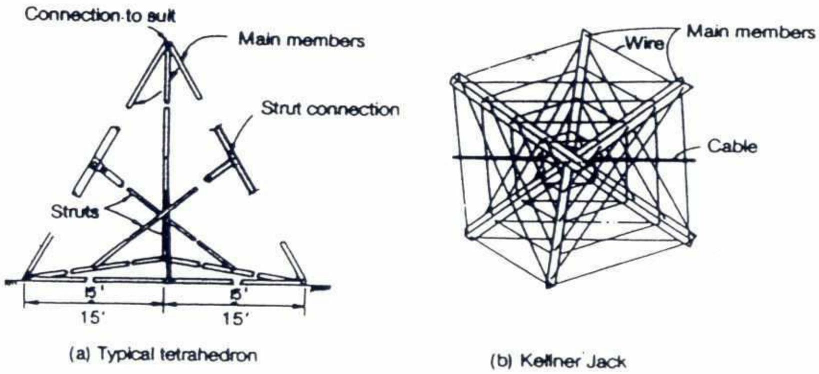


Fig. 3.3-31: Steel jack

(Source: Chang, 1988)

(ii) Cows

For arrangements of cows reference is made to Fig. 3.3-32 and 3.3-33.

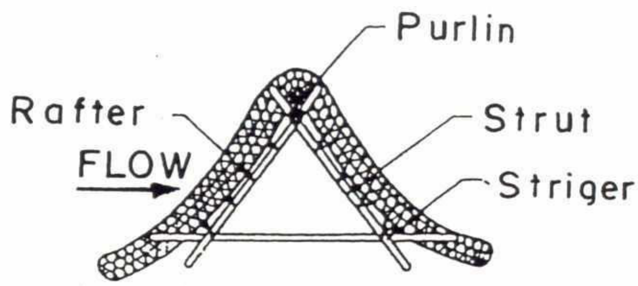


Fig. 3.3-32: Stabled Cow

(Source: UN, ECAFE, 1953)

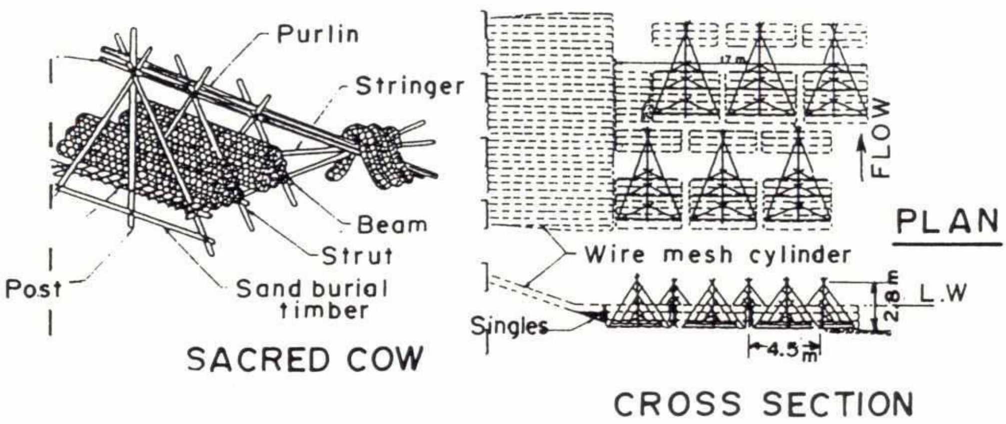


Fig. 3.3-33: Arrangement of sacred cow

(Source: UN, ECAFE, 1953)

(iii) Porcupines

Porcupine is a kind of frame made from bamboo and filled with brick bats. Porcupines are placed on river banks in rows in a staggered way. They are quite cheap but labour intensive in construction and placement. They are effective in silt laden small channels and tidal rivers as has been found in Satkhira and other places in Bangladesh.

Their use in flushy rivers like Teesta or big river like Jamuna were found to be not satisfactory in most of the cases. When the velocity of the river is high it is difficult to keep the porcupines in position because of higher drag force pushing the porcupines with the flowing water. Some constructional impression is given in Fig. 3.3-34.

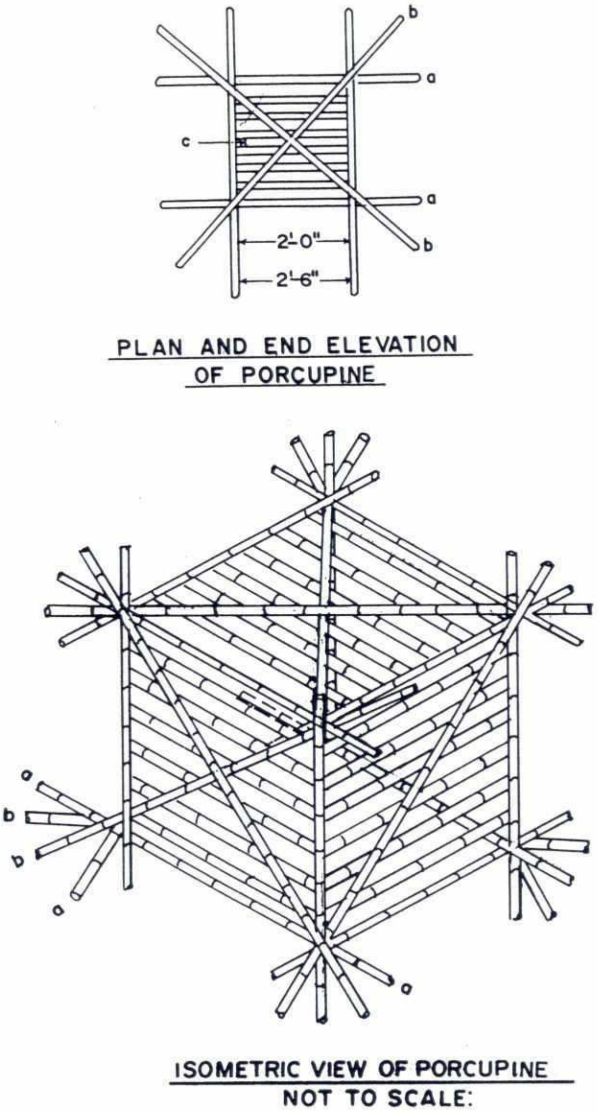


Fig. 3.3-34: Porcupine

(Source: BWDB typical drawing)

(iv) Retards

Where the flow velocity is moderately high and the river carries silt, "retards" or structures are built along the bank toe with the intention of reducing flow velocity and inducing siltation along the structure. (Ref. Page 9-12, Design Manual, Third FCD Project, BWDB, 1987).

Retards consist of a row of all kind of jacks such as:

- o Steel or timber jacks
- o Timber piling
- o A single or double line of steel pipe and galvanised wire mesh fencing
- o Steel, timber or concrete tetrahedrons.

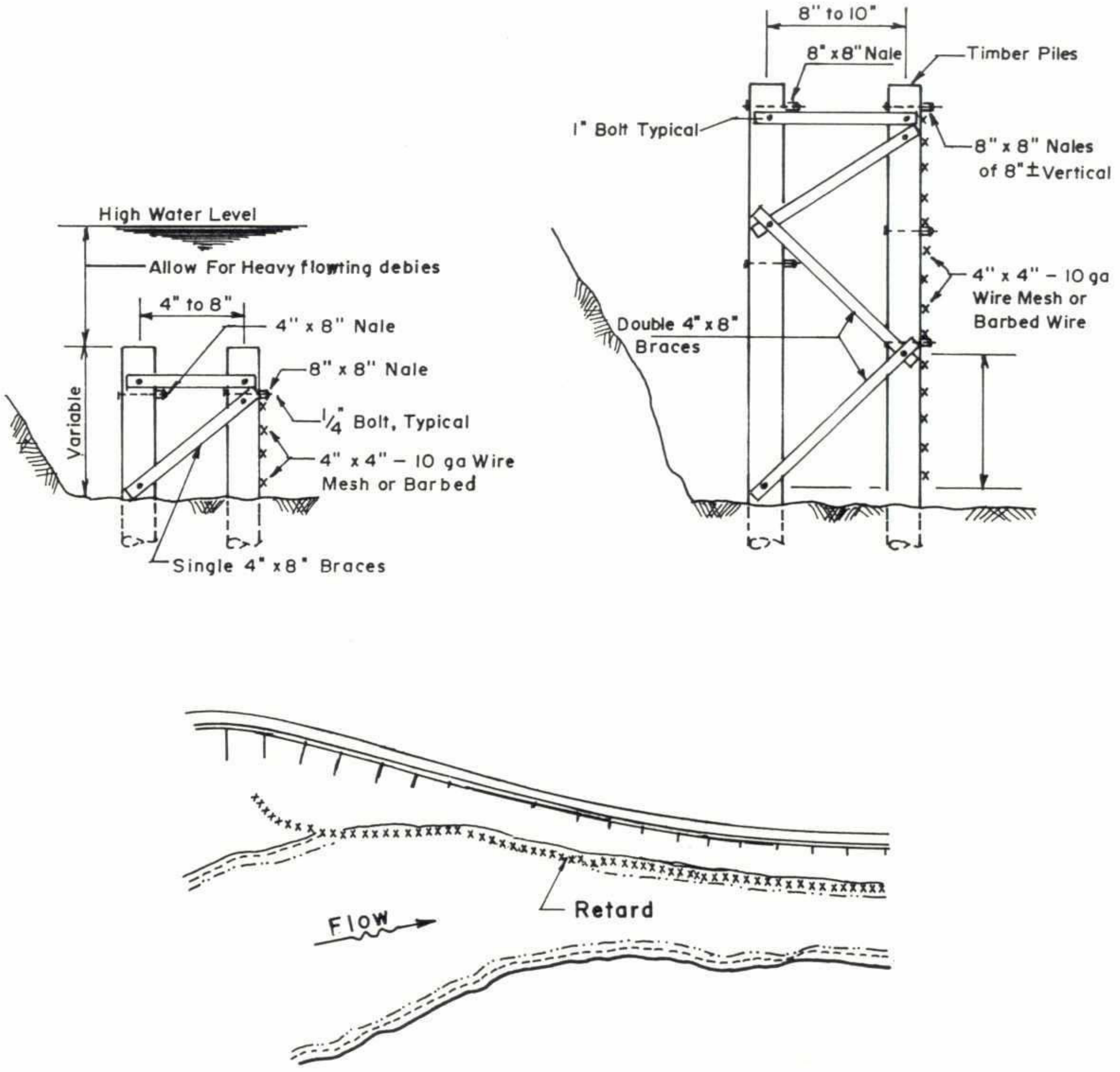


Fig. 3.3-35: Bank and shore protection in California Highway practice

(Source: Highway Division, Deptt. of Public works. California 1970)

4 REVIEW OF EXISTING STRATEGIES

4.1 INTRODUCTION

The starting point of the development of strategies are in general political decisions prioritizing the control and use of the river in a certain country. A possible sequence is for instance:

1. flood and erosion control (safety)
2. irrigation (food production)
3. hydropower

4. navigation
5. fisheries
6. recreation

These control and use functions are tackled from three angles
a) technical
b) economical
c) environmental

Also here political decisions determine priorities. It is clear that the first mentioned list of priorities is more fixed than the second list, especially because the environmental aspects, being long time neglected, are gaining priority.

The FAP 21/22 project is dealing with the development of measures for erosion control. The Project will select the measures on the basis of technical, economical and environmental considerations. A possible combination of measures along the river would be called a strategy.

In general the timing of the construction of measures and the hydraulic and morphological effects play an important role. A strategy is often considered far-field, covering a substantial part of the river, and both short and long term effects are important.

A combination of measures means that a mix is composed of various measures of both permanent and recurrent nature. The initial mix will be adapted as a function of time.

The timing is related to the duration of the construction per measure, the sequence of the construction, etc., hence, the overall planning of the strategy. Phasing is also important to reduce negative initial effects, such as a rise of water levels for instance due to measures narrowing the channel. These effects are fading out due to morphological processes, scouring in this case. Consequently timing of large scale implementation should be attuned to the morphological time scales.

A complicating factor is that the autonomous morphological processes, being already hardly predictable, are affected by the impacts of the measures. But the reverse is also true : the impacts (or effectivity) are affected by the river behaviour, see the example in Section 3.3. This interaction further frustrates the predictability of the final results. It explains why, especially for the dynamic alluvial rivers, even nowadays trial and error plays an important role in the set-up of a strategy.

River training activities have often been developed in the course of centuries, new targets and new insights leading to a new strategy. Hereafter, mainly recent existing strategies are emphasized. Some results of strategies with respect to a number of European rivers (Rhine, Rhône and Weser) as collected from literature, have been summarized in Section 4.2. Next to this a summary of the findings from the Study Tour to China and the United States is presented in Section 4.3. The Study Tour Report contains information as collected in the field and from extensive discussions with river authorities, technical staff, etc. with respect to:

- Yellow river and Yangtze Kiang river (China)
- Mississippi river (United States)

With respect to strategies the present "State-of-the-Art" report will be mainly based on this very Study Tour.

The river training strategy applied for the Zaire river in Africa should be mentioned here as it concerns active intervention in a morphological active braided river (Peters, 1988). The strategy is more directed to navigation channel improvement than to bank protection and makes use of dredging as the main measure. Through a method to predict morphological changes it is tried to keep dredging efforts at an economically acceptable level. However the average sediment concentration of 40 mg/1 is much less than the concentrations in the Jamuna river. Therefore, the dredging strategy seems more suitable for the Zaire river than for the Jamuna river and will not be further elaborated in this report.

4.2 STRATEGIES IN EUROPE

Reference is made to the extensive review on "Historical Change of large alluvial rivers in Western Europe" (see Petts 1989). From this review a summary based on the interests of the present "State-of-the-Art" is given in this section. This summary gives some details on river training of the rivers Rhine, Rhône and Weser.

o The Rhine river (Germany, The Netherlands)

Since the dawn of written history river training works have been carried out along the Rhine. However these works were executed initially very locally protecting a town or a traffic control post; e.g. by the Romans about 2000 years ago.

In the 17th century people started already with cut-offs of river beds (Petts 1989, p.71). But is was only in the previous century that the administrative structure developed so far that the strategies could be implemented, e.g. by G Tulla on the upper Rhine.

In the Lower Rhine the main objectives of river training were appropriate distribution of flow in the delta and improvement of the navigability. The first important works started in the 17th century. When floods continued to be hazardous, flood protection became also a main objective, which initiated in the 19th century a number of organisational activities on a governmental level in order to formulate a final solution of the inundations. Due to these chronological developments the strategy for the lower Rhine can be considered as a chain of cut-offs and confinements with groynes. These works were carried out in steps over various decades as shown in the Fig. 4.2-1 and 4.2-2 (Petts, 1989).

Nowadays, due to the increased importance of the environmental aspects, the measures to meet the various objectives of the strategy are changing. Important subjects tackled in the pre-design process of river training works are for instance:

- ecological aspects of river waters, e.g. currents and shelter in view of breeding places for fish, etc.
- natural values of the flood plains and levees

These subjects lead to preferences for

- under-water measures (no rip-rap up to HWL) and cribs instead of high crested groynes
- no disruption of the flooding characteristics of the flood plain
- revetments with vegetation creating natural looking banks.

For recent strategies in the lower Rhine, reference is also made to Swanenberg, 1988.

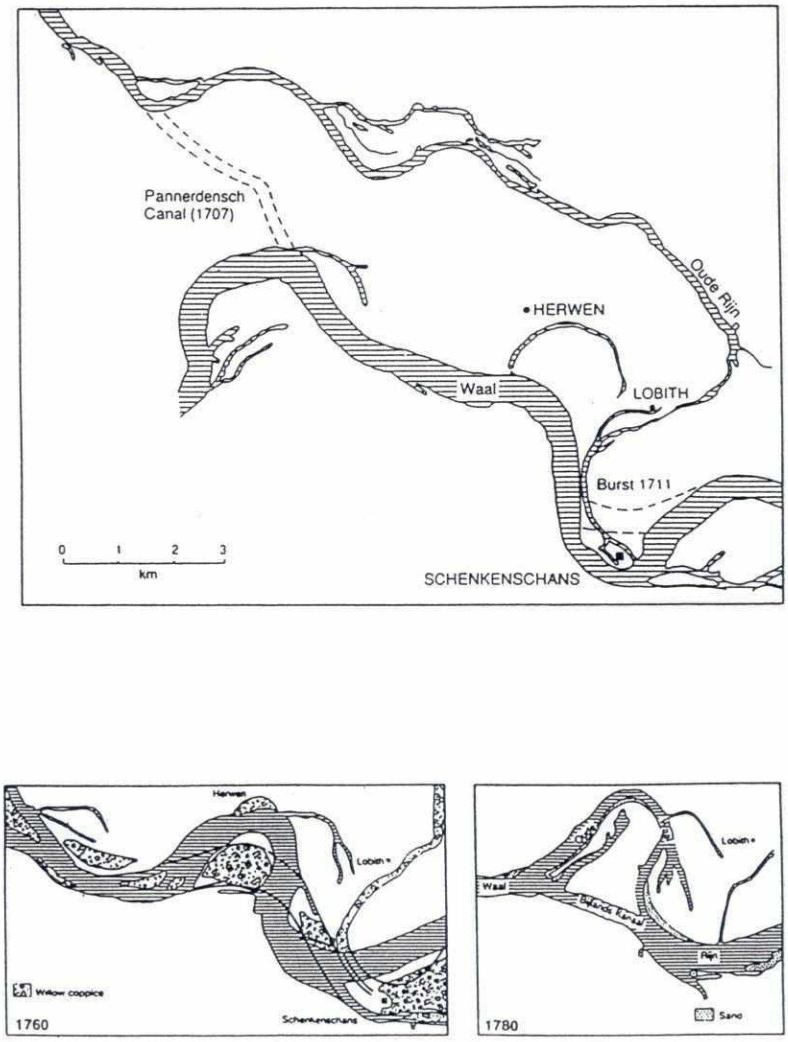
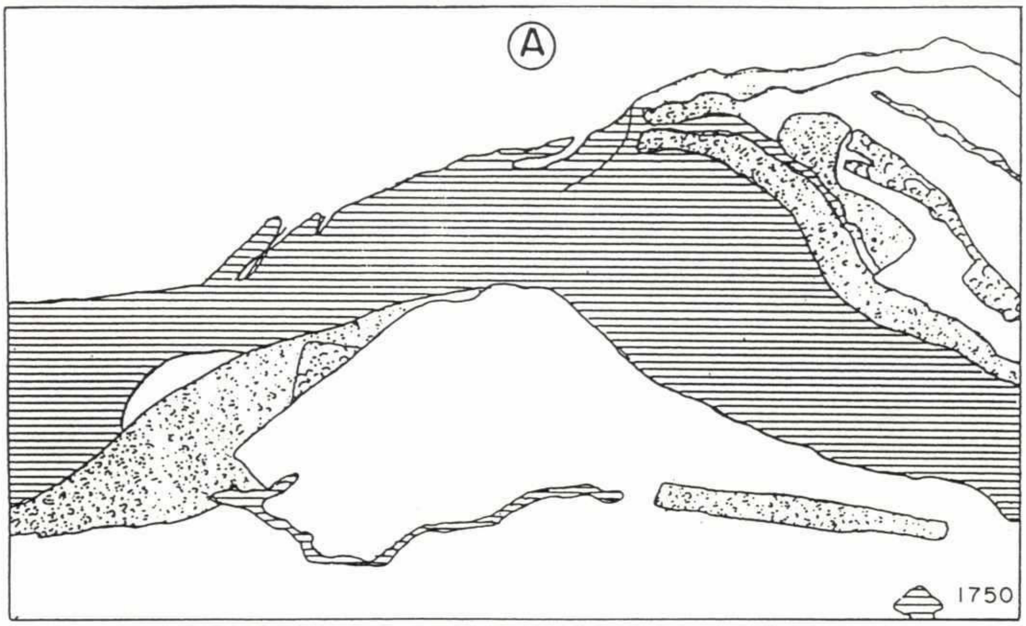


Fig. 4.2-1: The situation near the first Rhine bifurcation at the end of the 17th century above, and, below, the Waal meander near Herwen around 1760 (left) also showing the situation after the construction of the Bijlandsch Kanaal around 1780.

(Source: Petts et al., 1989)



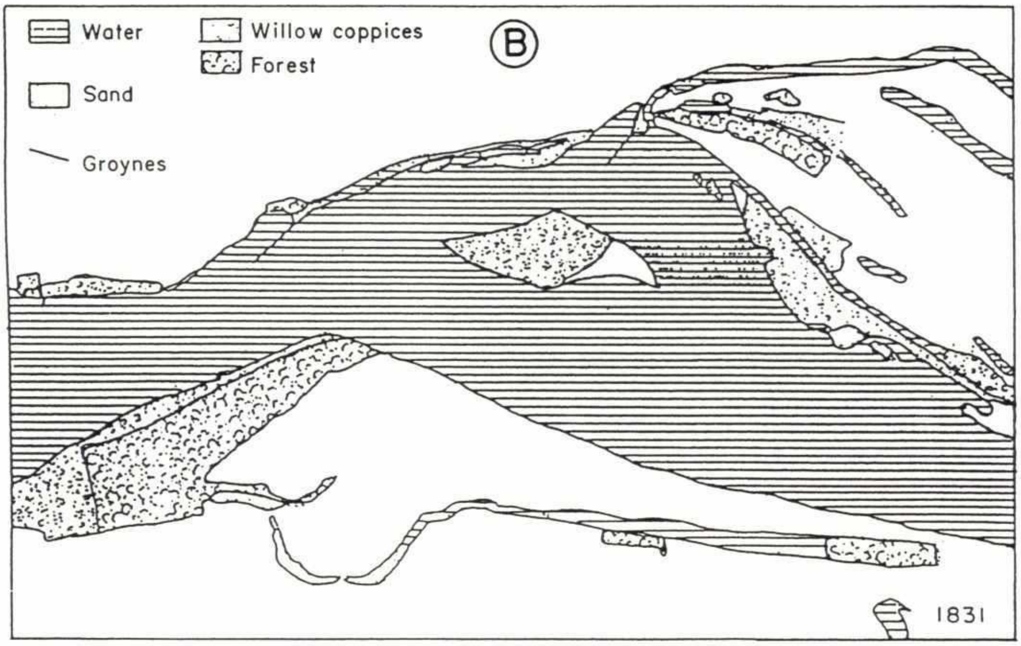


Fig. 4.2-2: The Waal river near km 899-901 showing channel changes over a period of about 200 years. About 1830 (Fig. B) the main channel had been diverted southward, a sand bar had developed between the two channels, and some dyke sections were protected by groynes. By 1870 (Fig. C) the northern channel had been dammed. The dam is protected by short groynes; and

(Source: Petts et al., 1989)

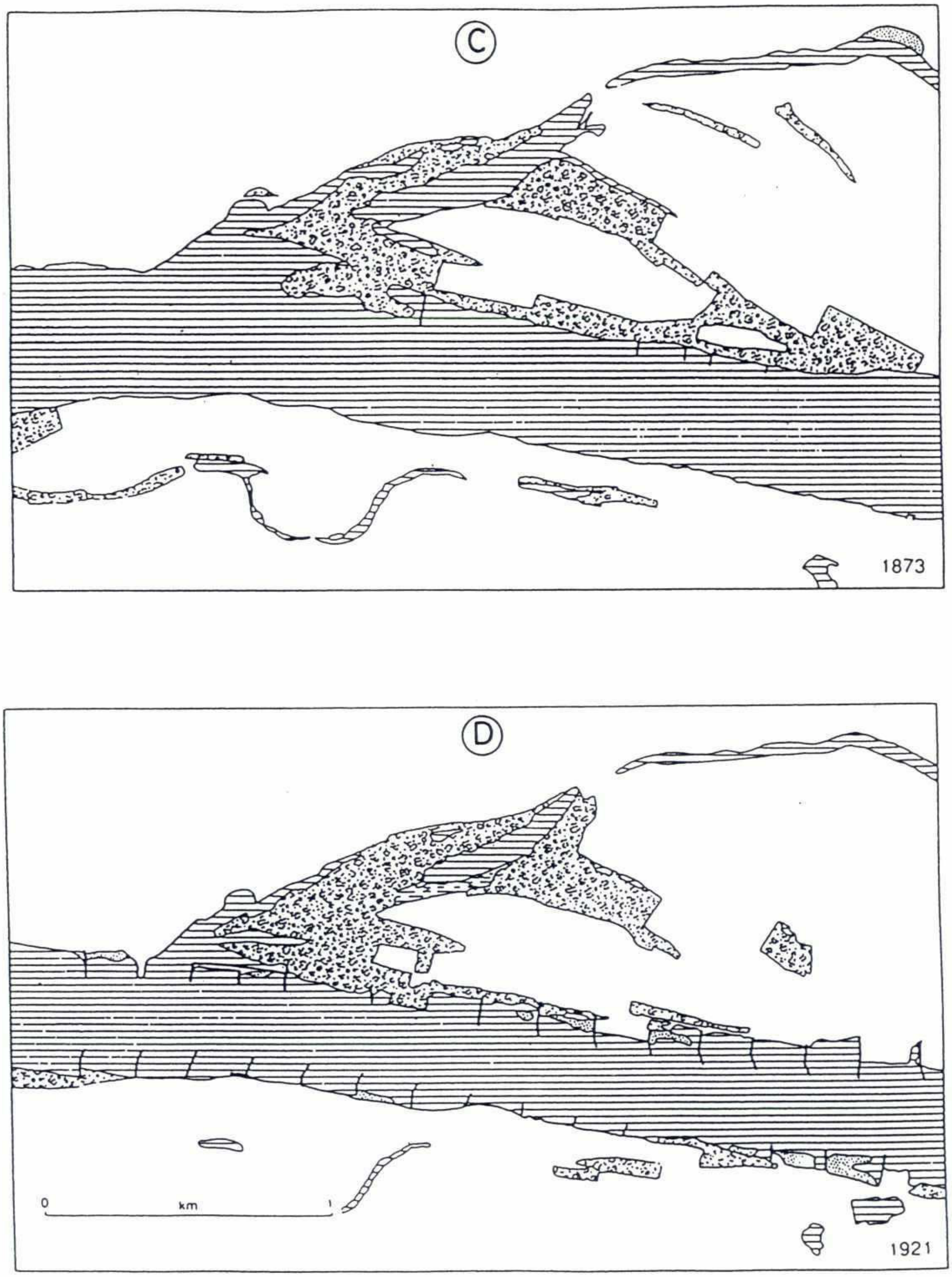


Fig. 4.2-2 (continued): The southern bank has been straightened. Behind the dam, large areas have been silted up and are occupied by willow coppices. By 1920 (Fig. D), channel constriction by groynes had been completed. The open water area had decreased and older willow coppices had been removed, but new willow coppices had developed in areas recently silted up.

(Source: Petts et al., 1989)

o The Rhône river (France)

The Rhône river represents a good example of a large European river that has been influenced by a range of human influences over several hundred years. In contrast to the small watercourses that were drastically modified by human activities during the last thousand years because available technologies allowed such transformations, in the case of large rivers major impacts are associated with the last three centuries. The most ancient modifications of large rivers were probably military ones and rather slight in comparison with what was to follow. Real impacts are due to civil engineering works implemented to promote the following three main objectives:

1) facilitation of navigation
2) protection of towns and lands against flood
3) production of hydroelectric energy.

The main and direct consequences of harnessing the river have been channelization, constriction of the active alluvial valley and discharge controls. These consequences will be indicated briefly with the following example. Along the Rhône river, engineering works began at the end of 1840 when Lyon tried to promote steam shipping on the Rhône and Saône rivers upstream of the town. It was then necessary to build stone dykes in order to concentrate low flows to create a unique deep channel. On account of the lack of public funds many dykes were built. These low-level dykes blocked only the upstream end of side channels at low flow; at high discharges they were submerged as water overflowed the dykes rejuvenating the network of minor channels. In the area of Brégnier-Cordon this type of embankment was constructed during the 1880s and their impact lasted until the recent construction of the hydroelectric development schemes. A chronological impression of this impact is shown in Fig. 4.2-3.

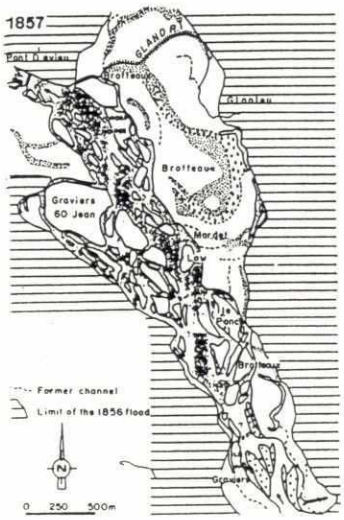


Fig. 4.2-3: Lay-outs of the Brégnier-Cordon sector showing the numberous braided channels within the aggrading central area and the anastomosed channels in the lower lateral belts

(Source: Petts et al., 1989)

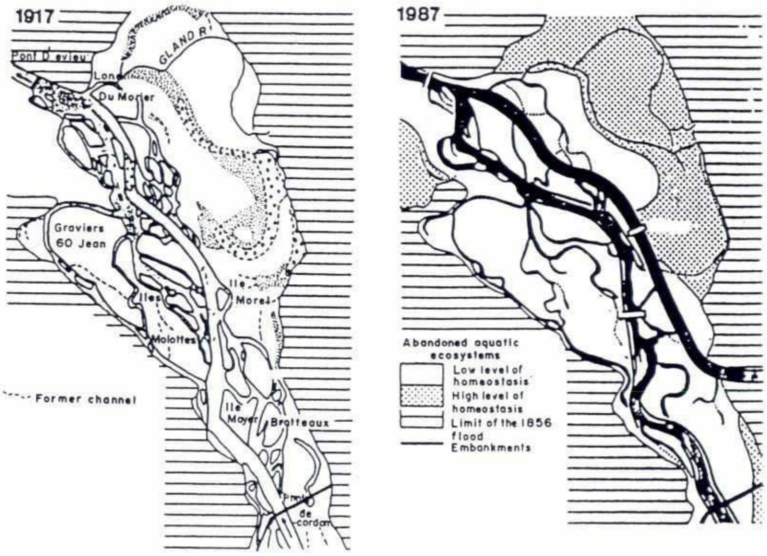


Fig. 4.2-3 (continued) : Lay-outs of the Brégnier-Cordon sector showing the numberous braided channels within the aggrading central area and the anastomosed channels in the lower lateral belts

o Lower Weser River (Germany)

The main objective to channelize the Lower Weser River in Germany was the stimulation of the economical expansion by facilitating see-going navigation. This will be described briefly as an example of a strategy.

Today the Lower Weser is one of most regulated rivers used as an ocean-going shipway in Europe. The first attempts in regulation go back to the 17th century. The depth of the Unterweser decreased heavily after the middle ages, due to increasing erosion upstream and the initiation of dyking in the estuarine area: in 1750 its depth was only 0.8 m downstream of Bremen. To ensure economic competitiveness of the harbours of Bremen, some attempts had been made in the 18th and 19th centuries to deepen the river-bed. However, they were of limited success. The first deepening showing substantial results was done from 1887 to 1895. Franzius (1895), the leading engineer used three main principles which are still common:

1. Shaping the Unterweser into a funnel to increase the tidal exchange
2. diminishing river channel bifurcation and
3. concentrating the tidal currents to the navigable channel by constructing embankments, groynes, etc.

Tidal exchanges and tidal currents increased, deepening the river-bed, because the tides intruded much further up the river. In addition, intensive dredging activities supported this process. Subsequently, seagoing vessels drawing 5 m, could reach the harbours at Bremen.

The same principles were used during four further phases of channel deepening and dredging: 1913-1916 to 7 m; 1925-1928 to 8 m; 1953-1959 to 8.7 m; and 1973-1977 to 10.5 m (called 9 m deepening due to the changed reference level: sea-chart zero).

This sequence of engineering works drastically changed the morphological, hydrological and ecological conditions within the Lower Weser:

1. The tidal range at Bremen increased from about 0.3 m around 1880 to 4 m today and altered littoral habitats.
2. The river surface as well as the riparian area were reduced to about one third (Fig. 4.2-4). Most of the ecological highly valuable backwaters and reed and mud-flats were lost by filling with dredged material. At the same time dykes intruded forward into the navigable channel. Today, nearly no adjoining marshes and wetlands are to be found.

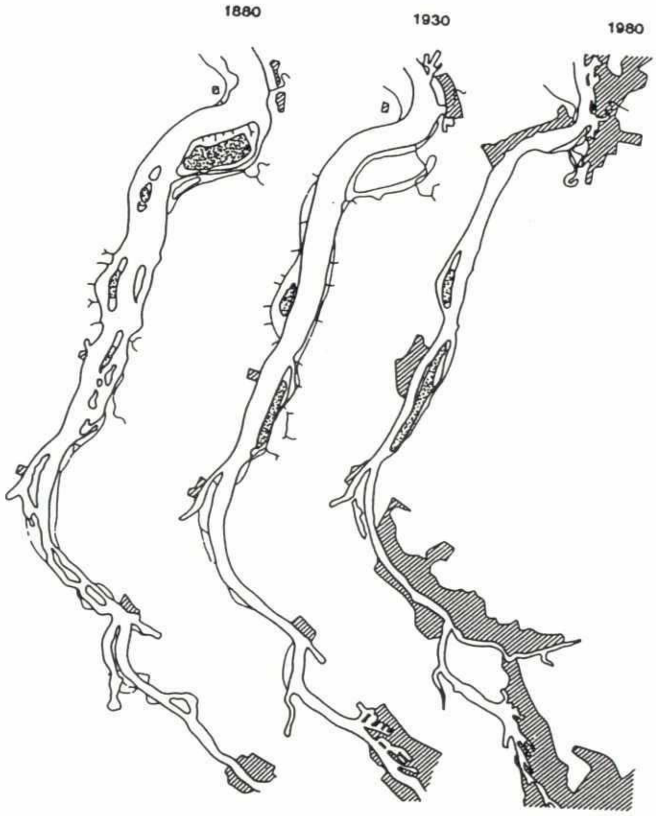


Fig. 4.2-4: Reduction of the Lower Weser and after channelization around 1880, 1930, and 1980.

(Source: Petts et al., 1989)

3. About 60 percent of the littoral embankments of the navigable channel are covered by various packing materials, less than 40 percent are left uncovered. Lotic zones and submerged plants are virtually missing.

4. The river profile was changed into a channel-like one and, thus, became very monotonous, due to the extreme deepening operations (Fig. 4.2-5).
5. In 1912 a tidal barrage at Bremen-Hemelingen was built to stop erosion by the falling low water levels. The barrier interrupted spawning migrations of some fish species and caused a decrease of population densities.
6. Due to strong siltation, especially in the tidal harbours of Bremen, (siltation rate 1981-86: 23.2 cm yr^{-1} or about 355000 m^3 yr^{-1} (Hafenbauamt, 1987), much maintenance dredging continues. The dredged material (mainly silt) had to be disposed of, because of its high pollution load.
7. On the other hand the Lower Weser regulation was the key to a developing the small town of Bremen into an important industrial region.

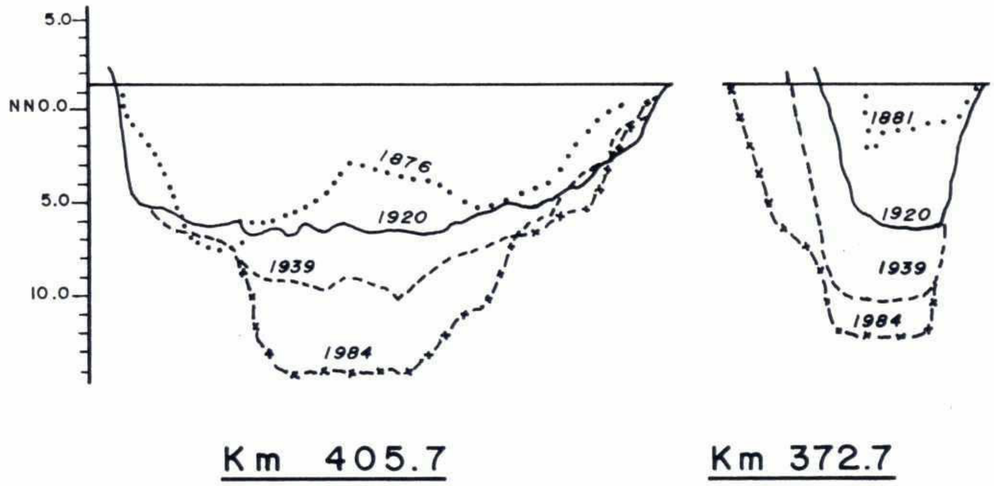


Fig. 4.2-5: Profiles of the Lower Weser at Brake (km 405.7) and Bremen Harbour (km 372.7) during different stages of channelization.

(Source: Petts et al., 1989)

The described process shows again that the efforts to train a river mainly is based on a process of trial and error and includes contradictory objectives.

4.3 BRAIDED RIVERS STUDY TOUR TO CHINA AND UNITED STATES OF AMERICA (USA)

To make use of the experience gained in other countries on comparably large braided rivers, participants of GOB and the Consultant made a study tour to the Yellow and Yangtze Kiang rivers in China and to the Mississippi river in USA. The impressions obtained by own experience and the information gathered in intensive discussions with representatives of the respective river commissions gave valuable input to the Project, particularly referring to experiences in training braided rivers. The experiences are reflected in many aspects of the present report.

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Additionally the main findings and conclusions are summarized below.

Experiences from the Yellow and the Yangtze Kiang rivers

- Long history of human interventions in the natural behaviour of mighty rivers of Yellow and Yangtze Kiang are available. Such knowledge suggests that flood control measures are slow processes of learning the river and gradual process of adjustment: a true trial and error approach towards taming the river. Any major interventions in a big river might have very adverse effects.
- Experiences from Yellow and Yangtze Kiang rivers suggest that no natural "nodal points" in a river appear do exist, which may be termed as a "hard points" in time scale. Rather, in a location on a river bank, if a place could be made a "hard point" by any engineering works, then this could be treated as a "nodal point" to fix up other hard points in the river alignment.
- Chinese advise for Jamuna is not to try for a quick solution. As to the complexity, the Jamuna offers a much difficult problem to solve than either of the Yellow or the Yangtze Kiang. However, they believe that a "hard point" concept should be applicable to the Jamuna.

Experiences from the Mississippi river

- Long recorded histories are available for flood control & river stabilization for the Mississippi river. They suggest that there are no text book approach to the solution of such problems. By trial and error, through methods developed on mistakes and failures the river was brought to the present state of stability.
- A general master plan is to be developed based on a general consensus on national objectives and goals. Implementation then should proceed at per necessity and dictated by the resource availability within the broader frame of the Master Plan. Necessary technical, administrative and financial systems should be developed to respond on an annual cycle.
- That implies special administrative structures which allow for both a long term development and a quick response at the same time.

Differences to the Jamuna river

Some important differences between the visited rivers and the Jamuna have to be taken note of:

- River training measures were taken up after a fully operable flood protection by retired embankments. In fact, generally the first river training measures consisted in protection of endangered flood protection embankments.

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- Due to their higher developed economics along the river the economic returns for flood and bank protection works are much higher than anticipated for the Jamuna. Whereas figures in China are questionable because of their economic system, a benefit; cost ratio of 39:1 was mentioned for the Mississippi works.
- The Jamuna area is much shorter on high quality construction material such as rock, rip-rap etc. and costs for them would be very high.

No recurrent measures like bandalling, bottom vanes etc. were reported of on any of the rivers visited. All measures implemented are to be considered as heavy and capital intensive ones.

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PART B

PRELIMINARY SELECTION OF RECURRENT MEASURES

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PART B: PRELIMINARY SELECTION OF RECURRENT MEASURES

5 PRELIMINARY SELECTION OF RECURRENT MEASURES

In this chapter a preliminary selection is given of recurrent measures to be used for the erosion control. Out of the review of recurrent measures, presented in Section 3.3, measures are chosen which may be applied for the various methods of erosion control given in Section 2.3. Out of these possibilities which are described in Sections 5.1 to 5.3, a selection is made and presented in Section 5.4.

This selection is based upon a qualitative assessment of the effectivity and is therefore a preliminary one. After a more quantitative assessment of the hydraulic and morphological effects (Chapter 6) a more final selection can be given (based on hydraulic and morphological considerations only).

For a part of the recurrent measures considered hereafter, low cost solutions seem possible. The set-up of this chapter is indicated in the matrix according to Table 5-1.

Recurrent measures in channels	Redistribution of flow at bifurcation	Distribution of flow in outer channel	Direct protection at outer bank
Dredging	-	5.2.1	5.3.1
Revetments	-	-	5.3.2
Dikes	5.1.2	5.2.2	-
Vanes	5.1.3	5.2.3	-
Jacks	5.1.4	5.2.4	-
Artificial cut-off	-	-	-

Table 5-1: Set-up of selection

This chapter ends with a summary (Section 5.5) and suggestions for further verifications of the preliminary selection (Section 5.6)

Notes:

1. The hydraulic and morphological revetments and dikes are (at least during the lifetime of the structures) the same for both permanent and recurrent measures. Hence, the selection criterion applied here (the effectivity) does not lead to a preference for permanent or recurrent. This choice should be made considering time scales, design aspects and cost-effectivity.

2. This chapter emphasizes the vanes and jetties as these are the measures being exclusively recurrent.
3. The artificial cut-off has to be considered as an individual measure, which might be attractive with respect to AFPM. But to the specific features of this measure some details will be presented in Section 5.4.

5.1 MEASURES AT BIFURCATIONS

In the following sections the possibilities to redistribute the flow at the bifurcation with the aid of various recurrent measures are discussed. The notations of the flow at the bifurcation are common to those as used in Sub-section 2.4.1.

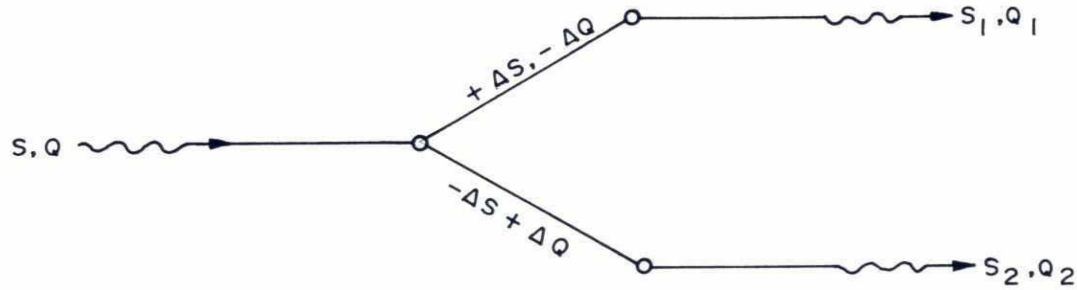

Fig. 5.1-1: Notation at bifurcation

<u>**Notes:**</u>

o channel 1 is the outer channel which should be tamed.

o ΔS and ΔQ are supposed to have positive values.

The determination of the effectivity of measures means here a qualitative and later a quantitative assessment of the impact of the measure on ΔS and ΔQ values. In other words: how will a measure affect the S_1/S_2 and Q_1/Q_2 ratios initially and as a function of time ?

5.1.1 <u>Dredging</u>

Basically dredging can be applied to redistribute the flow either by vast dredging enlarging channel 2 while diminishing channel 1 or by changing the configuration at the confluence.

The first approach is certainly not a low cost solution. In view of the required dredge capacity this approach seems only feasible in secondary or smaller channels. In a main channel the approach asks for a dredge capacity which is a multiple of the total dredge capacity available in Bangladesh (see Section 3.2.3 and DHV 1988 on IWT master planning).

The second approach with works at the confluence aims at reshaping the confluence to affect the flow distribution. Some possibilities are :

(i) Planform Correction

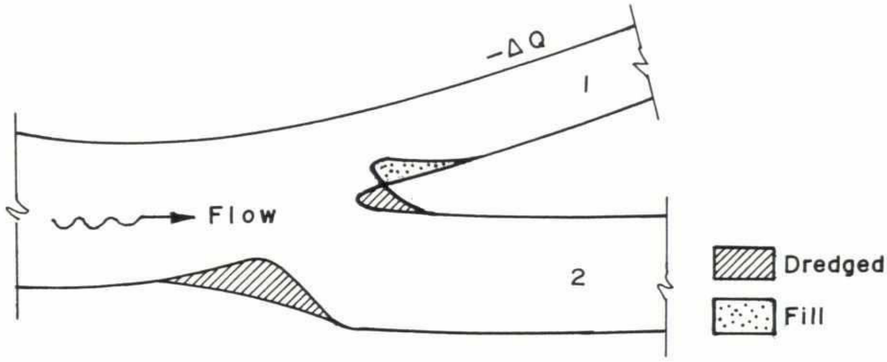

Fig. 5.1-2: Planform corrections

The principle of this method (see Fig. 5.1-2) is that the flow patterns at the bifurcation are affected in such a way that channel 2 is attracting more flow and channel 1 less. Moreover, the wet areas of the cross-section at both channel entrances are adapted, with similar result. This will lead to sedimentation in channel 1 and erosion in channel 2. Also this method is not a low cost one.

(ii) Bed Load Corrections
In principle the bedforms at the bifurcation may be reshaped in such a way that the bed load towards channel 1 will be increased. Reshaping may consist of dredging a deeper channel towards channel 1, narrowing and deepening the entrance of channel 1 and widening and shallowing the entrance of channel 2.

The main uncertainty here is how long a deeper channel may function in the very dynamic river bed of the Jamuna. Will it be closed by the first dunes passing by ? Anyhow the system is not a low cost solution.

(iii) Roughness Corrections
Dredging may also aim at changing the bedforms in the entrances either in one or in both of the channels. By increasing ripple and dune heights the hydraulic resistance in channel 1 may be increased. By flattening the river bed in the entrance of channel 2 the roughness there will be decreased.

In principle only the flattening in channel 2 can be done by low cost dredging techniques using e.g. a water injection dredge, a bed leveller or a mud wheeler. A dynamic maintenance service is required to tackle the crests of the passing dunes.

Though the system will most probably work in low flow conditions, doubts exist on the effectivity during bank full discharges. During low flow when water depths fall below 2 m, removal of say 0.5 m by low cost dredging activities are significant for cross-sectional area and roughness. But what will be the impact during say mid-to flood-flow conditions with water depths of 20 m or more ?

5.1.2 Dikes

For the adaptation of the flow distribution at the bifurcation with the aid of dikes, quite some possibilities exist. A few are indicated hereafter

(i) Impermeable Groynes

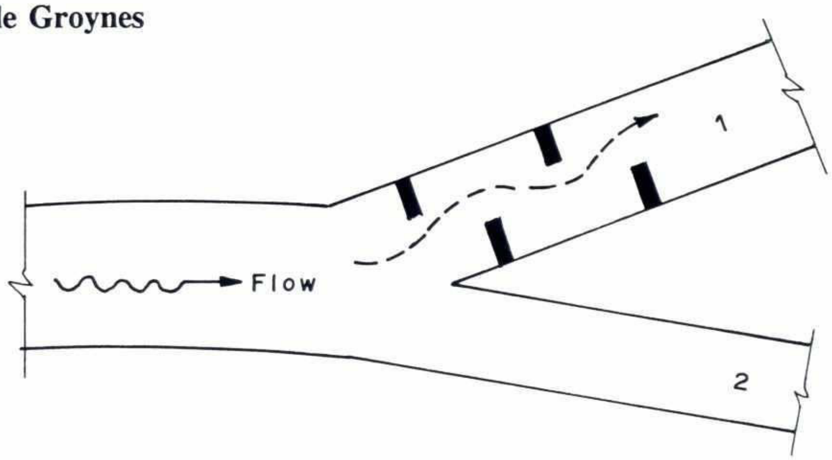

Fig. 5.1-3: Groynes in outer channel

Initially the groynes give additional roughness generating a ΔQ. The result is erosion in both channels. If the erosion process in channel 1 (see Fig. 5.1-3) is faster than in channel 2 the ΔQ value is decreasing. On the other hand a deeper channel through channel 1 may attract more bed load.

Other possibilities are applying groynes in the upstream part of the bifurcation, for instance as follows:

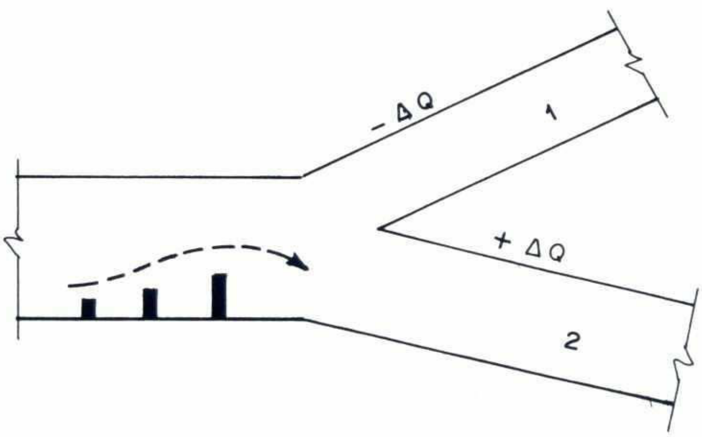

Fig. 5.1-4: Groynes in the upstream channel

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The aim of the lay-out as presented in Fig. 5.1-4 is that the current is accelerated first and rotates thereafter in the deceleration zone around the last groyne smoothly into channel 2, thus increasing the flow there. Proper functioning is only possible with a carefully designed lay-out.

(ii) Permeable Groynes

One of the permeable groynes is the needle groyne which can be used at several lengths and heights, with as a maximum a full screen in the entrance of the outer channel. If navigation permits, a screen at full length is attractive, as excessive scouring at groyne or screen head is avoided.

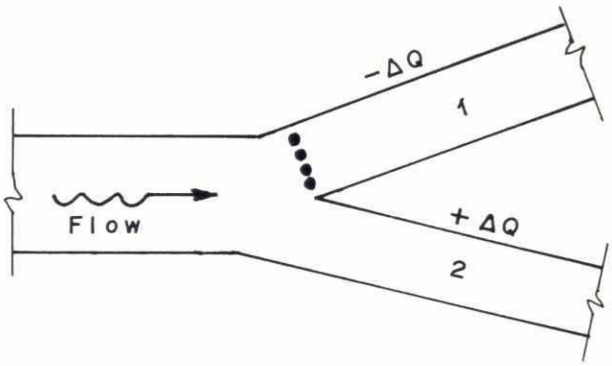

Fig. 5.1-5: Open pile screen

Initially, the additional resistance will reduce the flow (both water and sediment) in channel 1. The resulting sedimentation in channel 1 and scouring in channel 2 will amplify the initial effect. The effect depends mainly on the size of the blocking factor N*D/B

in which

N	=	number of piles
D	=	diameter of piles
B	=	length of the screen which equals in this case the width of entrance of channel 1.

With low blocking factors new equilibriums may occur as soon as the channel profiles and flows are attuned to each other. With higher blocking factors an ongoing process may be generated leading to a complete closing of channel 1. As sedimentation starts behind the screen progressing downstream, a more downstream position of the permeable groyne (just upstream of the eroding bank) is to be considered.

(iii) Cribs and Sills

Bottom cribs are the lower type of groynes. Applications are rather similar. Rapid increase of applications is noticed nowadays. The sills are in fact long bottom cribs crossing the river. Mostly they are applied in a permanent form e.g. just downstream of bridges, thus reducing scouring depth.

The effectivity (e.g. redistribution of flow at the bifurcation) depend amongst others on the location in the channel, which can be concluded from the study tour to China (see Section 4.3). At the moment of the study tour model tests were being performed in order to define an optimum location of the sill for a comparable situation as presented in Fig. 5.1-6 in the Yangtze Kiang river.

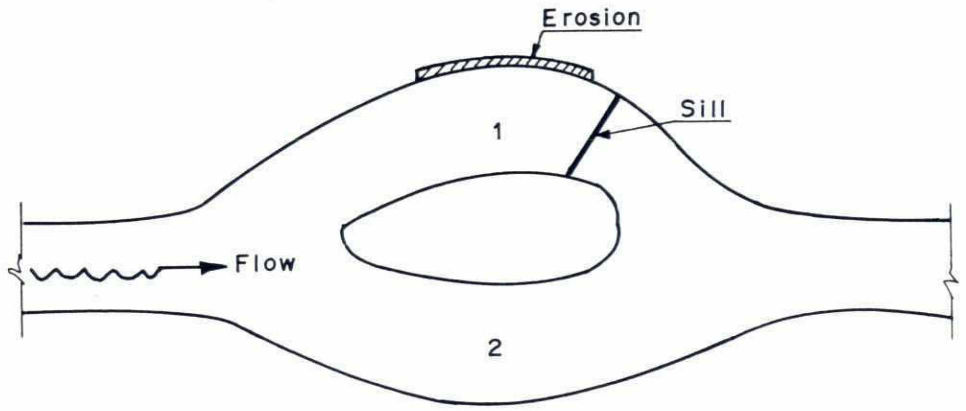

Fig. 5.1-6: Sill

The sill affects the flow distribution at the bifurcation (additional resistance) and causes sedimentation in the section with bank erosion. It is probable better to position the sill more downstream in the outer channel, see Section 5.2.2.

5.1.3 <u>Vanes</u>

The redistribution of the flow near the bifurcation can be realized with various type of vanes

(i) Bottom Vane

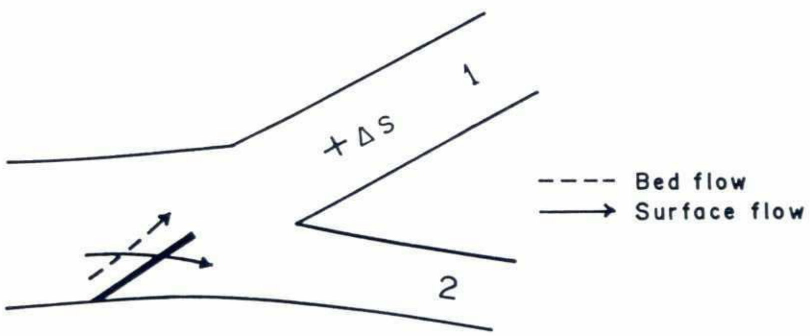

Fig. 5.1-7: Bottom vane

The bottom vane (or panel) (see Fig. 5.1-7) is installed somewhat upstream of the bifurcation to reduce the negative effect of the additional resistance in channel 2. The pre-rotation of the surface flow towards channel 2 reduces the above mentioned negative effect so the resulting consequences for the water flow are small. The main effect is that the lower part of the flow with the highest sediment concentrations is directed towards channel 1. Consequently the resulting effect will be $+\Delta S$.

Obviously more than one vane can be used next to each other in a row perpendicular to the current. A crucial point is here how long a bottom vane would be functioning on the dynamic river bed of the Jamuna.

(ii) Intermediate Vanes

Intermediate vanes are often composed of a series of foils located in a row. Basically they work as deflectors slightly changing the flow direction. In a limited cross-section this causes a spiral flow. The direction of rotation depends on the angle of attack and the vertical position of the vane.

Near the bifurcation they may be used as deflectors only to improve the inflow to channel 2 (see Fig. 5.1-8).

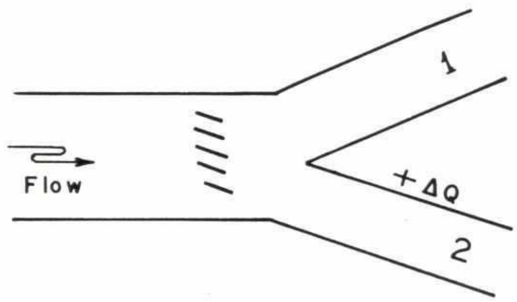

Fig. 5.1-8: Intermediate vane

(iii) Surface Vanes

Surface vanes either floating or rigidly installed on a frame (a bandal) can be applied in various ways. The bandal can be placed in the upstream section of the bifurcation similar to the lay out of the intermediate vane, see Fig. 5.1-8 above. However, the bandal can also be placed before the entrance of channel 1, as indicated in Fig. 5.1-9.

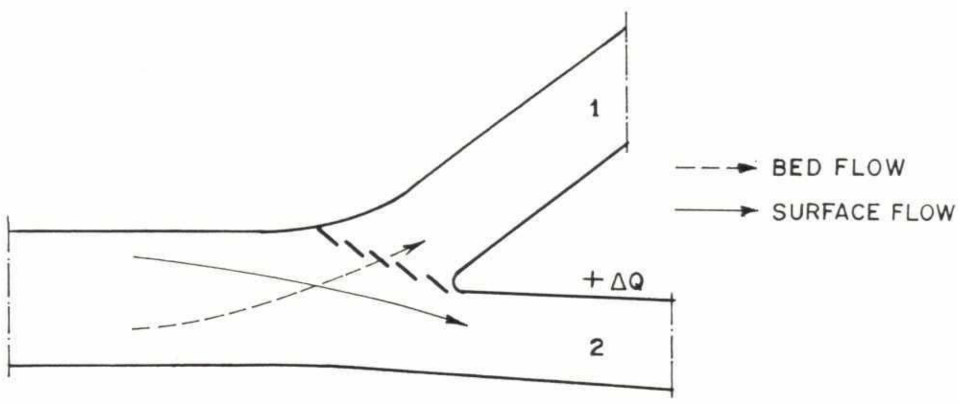

Fig. 5.1-9: Bandal

The bandal may consist of quite a number of screens (suits) or one screen. Also the length may vary (shorter screen only covering a part of the channel entrance).

The expected effects are: additional flow towards channel 2 due to the resistance and the deflection of the surface flow. Moreover the bed flow with the higher sediment concentrations is directed towards the outer channel, as the bed flow tends to pass the screens perpendicularly. These expected effects are very promising. Disadvantages are :

- the only experience available concerns low flow conditions
- fundamentals (e.g. a design manual) do not exist

(iv) Variants

All kind of variants are possible, as indicated already above. Also combinations are possible e.g. a bandal before the entrance of channel 1, see point (iii) with a bottom panel just before the bifurcation, see point (i).

5.1.4 Jacks

At the bifurcation jacks can be used in a row in a similar way as described for the open pile screen, Section 5.1.2 (ii). Instead of single needles, clumps or jacks can be applied.

5.2 MEASURES IN OUTER CHANNELS

In order to affect the flow and the shape of the cross-section in the eroding section of the outer channel, a number of measures are possible as indicated hereafter.

5.2.1 Dredging

With dredging the shape of the cross-section (see Fig. 5.2-1) with the deepest part near the eroding outer bank can be changed (channel 1 is supposed to be curved) :

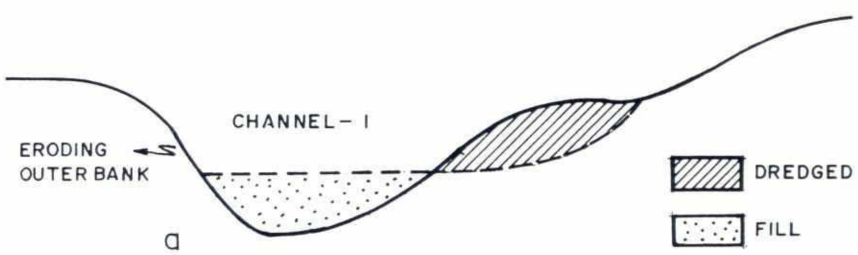

Fig. 5.2-1: Changing cross-profile

The reshaping will have the right effect on the horizontal flow distribution: the velocities near the outer bank will decrease and the velocities near the opposite bank will increase. This type of correction is also applied with agitation dredging. However, the situation is unstable. After the works, the spiral flow will resume eroding the channel bed near the outer bank, unless the bottom is protected there.

5.2.2 Dikes

The usual type of groynes and cribs can be used to keep the flow away from the bank and generate sedimentation in between. This is certainly not a low cost affair in main channels, even when recurrent methods (sand bags, etc.) are applied. An interesting variant is to apply a crib (or sill) in the downstream section of the outer channel, somewhere between the eroding section and the confluence (see Fig. 5.2-2).

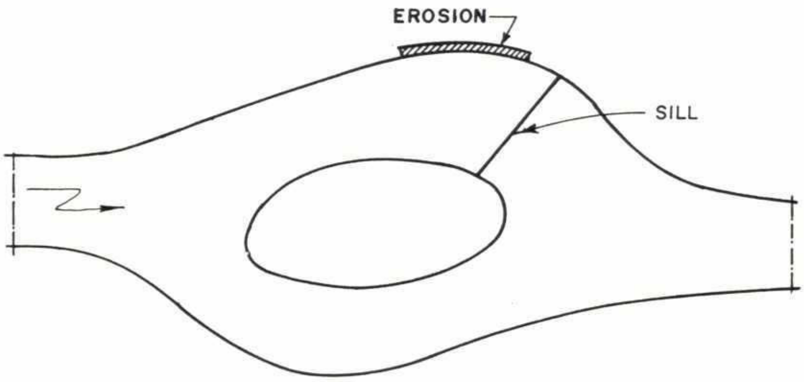

Fig. 5.2-2: Sill in outer channel

The sill is not only increasing the hydraulic resistance in the outer channel thus affecting the flow distribution at the bifurcation, but also generate sedimentation just upstream of the sill first, and gradually progressing in upstream direction. In this way the water depth at the toe of the eroding outer bank may decrease, reducing the scour.

5.2.3 Vanes

Some possibilities to reshape the cross-section in the outer channel using vanes are indicated below. The bottom vanes (see Fig. 5.2-3) are catching the bed load transport, directing it towards the outer bank. The surface flow, tending to cross the vanes perpendicularly is directed from the bank. For the Jamuna, the question arises if it is possible to install these vanes and arrest with them a part of the dunes passing by.

(i) Bottom Vanes

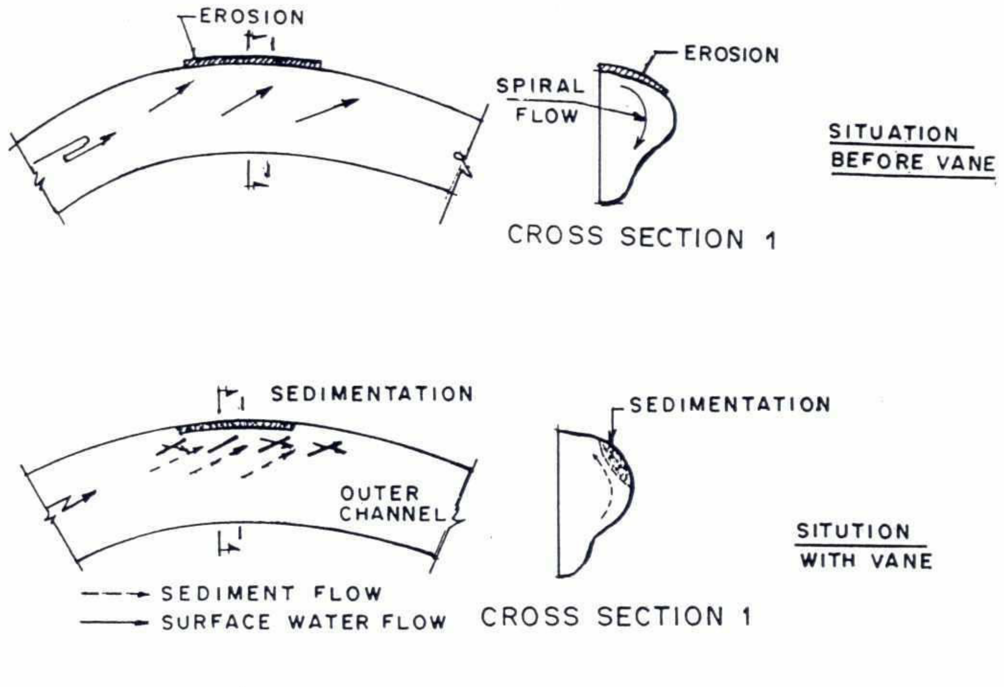

Fig. 5.2-3: Bottom vanes in outer channel

(ii) Surface Vanes

Obviously also bandals or floating vanes can be applied to adjust the bed flow and surface flow (see Fig. 5.2-4).

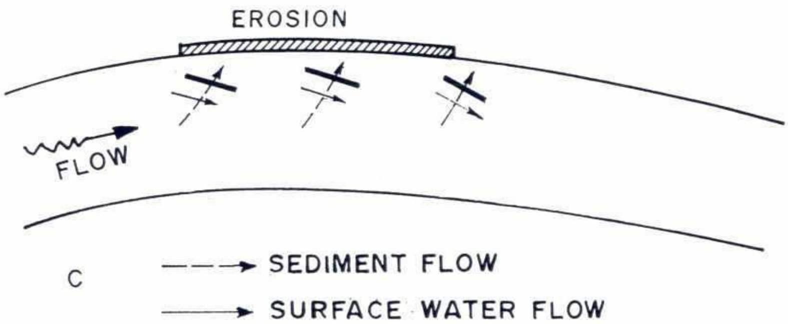

Fig. 5.2-4: Bandals in outer channel

The bandals cause sedimentation near the bank and redirect the surface flow towards the channel centre.

5.2.4 Jacks

Usually jacks are applied to redistribute the resistance in a cross-section. So from that point of view they are applicable here.

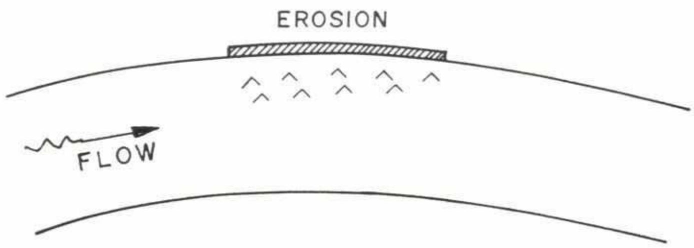

Fig. 5.2-5: Jacks near the outer bank

Obviously, in practice there will be a lot of problems to install the jacks in the deepest part of the channel.

5.3 MEASURES AT THE BANK

Measures to protect the outer bank directly against the eroding forces are either dredging or revetments.

5.3.1 Dredging

In fact this means that the outer bank is protected by a suppletion of sand covering the lower part of the endangered bank slopes (see Fig. 5.3-1).

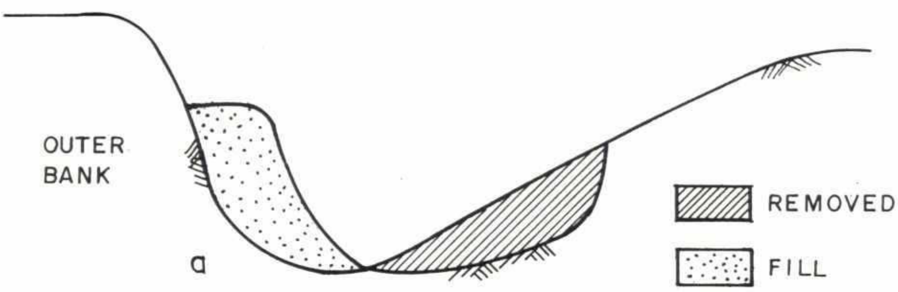

Fig. 5.3-1: Bank protection by suppletion

This way of bank protection on the Jamuna is probably not feasible. It is certainly not low cost. For the main channel the suppletion demand will exceed substantially the available dredge capacity. For example the situation is considered that dredging has to be carried out for a section with a length of 500 m, a depth of 20 m and a width of 1000 m. This should lead to a dredging volume of 10×10^6 m^3 which is much more than the available dredge capacity (see Table 3.2-3). Moreover, the final result is an unstable one.

5.3.2 Revetments

Revetments are the most direct answer to the erosion problems. However, the usual type of structures and the considerable slope dimensions lead to excessive costs. As long as major river sections are migrating the need for further developments of recurrent revetments remains urgent.

5.3.3 Permeable Groynes

Short permeable needle groynes along the eroding outer bank seem possible.

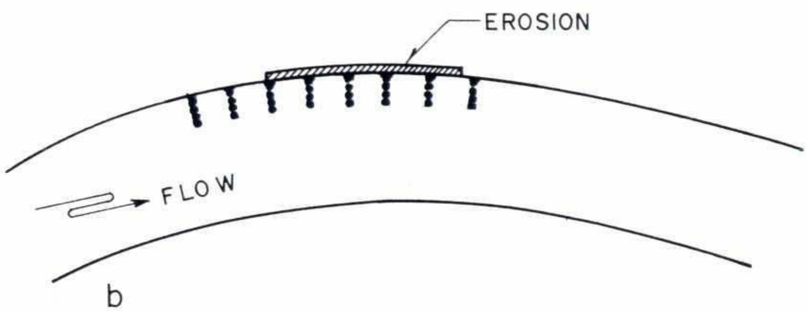

Fig. 5.3-2: Needle groynes at outer bank

5.4 ARTIFICIAL CUT-OFFS

This measure has to be considered as an individual measure with specific features of possible interest for AFPM. The section presents some interesting details regarding the artificial cut-off.

A cut-off can be defined as a process by which an alluvial river flowing through curves or bends abandons a particular bend and establishes its main flow along a new and comparatively straighter and shorter course. A typical example of a cut-off in a meandering river is shown in Fig. 3.4. In meandering rivers cut-offs do occur occasionally and they are considered to be the balancing mechanism for the increase in length of many meandering rivers due to bank erosion along the outer bends. Recently cut-offs were studied by Klaassen & van Zanten (1989) and criteria for the occurrence of a cut-off were developed by them. The discussion presented hereafter is mainly based on their analysis. In addition some results of a study by Biglari (1989) are presented here.

There is a major difference between a cut-off in a meandering river and in Jamuna type of rivers. In a meandering river the cut-off usually occurs through the neck of the meander, hence through floodplain soils. These floodplain soils are often cohesive, and a cut-off is initiated when the shear stress during floods exceeds the critical shear stress of these floodplain soils. Hence the criterion for initiation of a cut-of in a meandering river reads as:

$$\tau \geq \tau_{critical} \tag{5.4-1}$$

where τ = shear stress during flood, and τ_{critical} = critical shear stress of the floodplain soils, including the effect of vegetation if appropriate. The shear stress τ is the shear stress on the flood plain which corresponds according to Klaassen & van Zanten (1989) to:

$$\tau = \rho g h_{floodplain} \Psi i \qquad (5.4\text{-}2)$$

where ρ = density of water, g = accelaration due to gravity, $h_{floodplain}$ = water depth on the floodplain, ψ = cut-off ratio and i = river slope, defined as the ratio between the length of the cutoff bend and the direct distance over the neck of the meander. See Fig. 5.4-1 for a definition of the different symbols used hereafter.

It is obvious that the occurrence of a cut-off depends on the value of the water depth on the floodplain and ψ. Hence the larger the flood, the more bends will be cut-off.

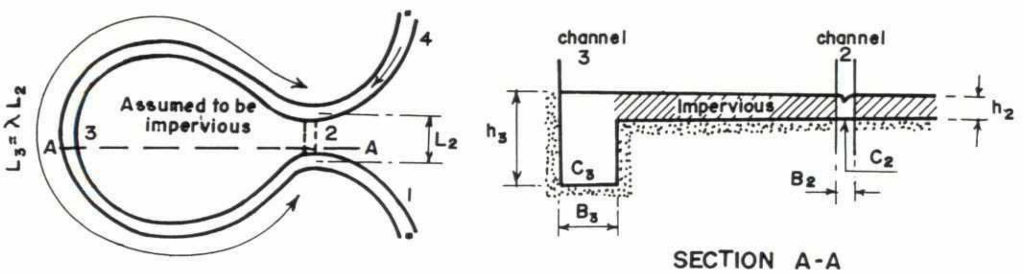

Fig. 5.4-1: Definition of cut-off ratio

For a braided sand bed river like the Jamuna river, different criteria are applicable. The major difference is that during floods everywhere the critical shear stress is exceeded. Whether or not a cut-off occurs, depends fully on the balance between the sediment transport entering a potential cut-off channel and the sediment transport capacity of this channel. This was elaborated by Klaassen & van Zanten (1989), in which design graphs were proposed. An important aspect appears to be the sediment distribution at the potential bifurcation. In the approach of Klaassen & van Zanten, this distribution is given by:

$$\frac{S_{2\ incoming}}{S_{3\ incoming}} = \frac{1}{\nu} \frac{Q_2}{Q_3} \qquad (5.4\text{-}3)$$

where ν = parameter which determines the distribution of the sediment at the potential bifurcation. The resulting design graph is shown in Fig. 5.4-2.

It follows from the above analysis that the occurrence of a cut-off for Jamuna type of braided sand bed rivers depends on the following parameters:

- cut-off ratio,
- relative water depth in the potential cut-off channel, where in the case of the Jamuna River the water depth h_2 has to be interpreted as the water depth over the chars and
- sediment distribution at the potential bifurcation.

The latter parameter is new compared to the criterion for meandering rivers. This distribution of the sediment at the bifurcation depends on the mode of transport (suspended or bed load) and on the geometry of the bifurcation (see ANNEX 11 for recent studies on the importance of this geometry).

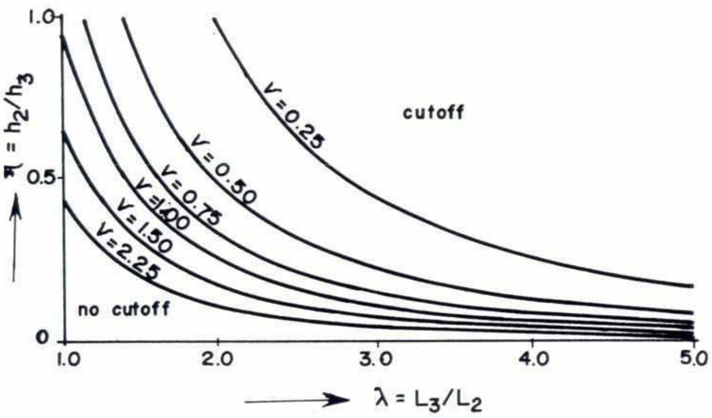

Fig. 5.4-2: Design graph for the occurrence of a cut-off for different values of ν

(Source: Klaassen & van Zanten, 1989)

The time-dependent development of a cut-off was studied by Biglari (1989). It was found that a typical time scale for the development of a cut-off can be derived which reads (see Klaassen & van Zanten, 1989):

$$T = \frac{\frac{B_2}{B_3} L_2 \, (h_3 - h_2)}{(S_{2 \; transport \; capacity} - S_{2 \; incoming})} \tag{5.4-4}$$

For definitions see Fig. 5.4-1 . It follows that a minimum value for T is found if $S_{2 \text{ incoming}} = 0$, but the time required can take much longer if $S_{2 \text{ incoming}}$ has a value. For $S_{2 \text{ incoming}} = S_{\text{transport capacity}}$ no cut-off will occur. It is clear that for a large water depth the cut-off will develop earlier than for a small water depth over the chars.

In Klaassen & Masselink (1992) some data on the occurrences of cut-offs in the Jamuna River are given. It appears that cut-offs develop very quickly for cut-off ratios between 1.0 and 1.6, whereas for meandering rivers the values of ν typically vary between 8 to 30. According to Fig. 5.4-2 a cut-off for $\lambda = 1.0$ implies that relatively sediment free water is entering into the potential cut-off channel. Hence cut-offs are a very powerfull mechanism in the Jamuna River.

It is clear that the occurrence of a cut-off can be accelerated by influencing the ratio η (= h_2/h_3). This can be done by dredging a pilot channel according to the alignment of the potential cut-off. The alignment of such a channel can only be decided upon after thorough study. It seems that the location of the bifurcation and the angle the potential channel is making with respect to the main flow direction is of critical importance (see ANNEX 11). It is not required to dredge the pilot channel at full depth nor at full width. In fact only a channel with fairly minor dimensions may be enough for the diverted flow to scour the cut-off channel up to the desired dimensions in due time. This holds for both the desired depth as well as for the desired width. For the dimensioning of pilot channels see Pilarczyk, 1990. Obviously the desired channel can be dredged at its final dimensions right away, but this will be very expensive. Within the frame-work of FAP 22 the interest is in letting the river do most of the work. The larger the pilot channel, the quicker the actual cut-off will have materialized.

5.5 SUMMARY

The most promising recurrent measures described in the previous sections are summarized in the matrix in Table 5.5-1.

Recurrent measure	At bifurcation	In outer channel	At outer bank
dredging	-	-	-
revetment	-	-	recurrent structures
groyne type	permeable groyne	sill	permeable groyne
vanes	surface vanes	bottom vanes surface vanes	-
jacks	-	-	-
bend cut-off	Short-cut of outflanking bends		

Table 5.5-1: Preliminary selection of recurrent measures

This preliminary selection is based upon the following considerations:

- o Dredging is always possible but is not selected in first instance. The effects of low cost dredging (LCD) are limited. Only in a few cases reshaping of a channel entrance or a cross-section using LCD techniques can be considered. Other types of dredging (e.g. cutter suction) is costly especially in the main channels (volumes to be dredged against available dredge capacity). Instead of dredging vast volumes, other measures using the considerable natural transport of fine sediments by the river are prefered.
- o Recurrent revetments at outer banks are possible; for instance with sand filled bags or tubes a mattress can be composed. Whether structural solutions can be found for the high banks of the main channels is questionable.
- o Permeable groyne types are preferable above impermeable ones, especially in view of scouring. For the same reason a needle groyne spanning the full width of the

channel is preferable above a groyne of limited length (scouring at groyne head). Also short permeable groynes may be considered to protect the outer bank. Sills are possibly applicable to reduce bank erosion in outer channels. The position of such sills should be downstream of the eroding area. A recurrent sill can be made of bags, tubes or sand or a combination thereof (e.g. protected sand dike).

- o Solutions with bottom vanes spread over a considerable width of the river are not selected because of the fast changing bed shapes. Only rows of vanes along the bank could be considered as a recurrent measure for the Jamuna river.
- o Solutions with groynes and bottom vanes which are only effective in a careful layout at the bifurcation are not selected, because a relative small change of the bed configuration may spoil the effects.
- o Surface vanes are probably effective because of the fine non-cohesive bed materials. This might be concluded from the application of bandals (surface vanes) for so many years in the Jamuna and such kind of rivers.
- o Jacks may be applied, especially in rows in the same way as the needle groynes. They are not selected but may be applied to replace needles for structural reasons.
- o As result of these qualitative considerations, it is decided that at this stage priority is given to those recurrent measures which aim at changing the flow conditions. The degree of these changes can be different. The measures may aim at either reducing the flow conditions (depth, velocities) towards more moderate values or reducing the flow up to zero (closing of the outer channel, eventually by repeating a couple of times the measures). The surface screens are recommended to be investigated in more detail in order to achieve a quantitative assessment of their effectivity.

5.6 VERIFICATION PROCEDURE OF MEASURES

The preliminary selection is based on a qualitative assessment of the effectivity. The first steps to be taken to verify the selection are:

- o checking whether no promising measures are missing (brain storm session)
- o checking whether the estimated effects (qualitatively assessed) are correct
- o trying to assess the effectivity of these measures more quantitatively.

The last two steps can be taken by executing first order computations. The computational frame work is given in Chapter 6 (simplified models). The mathematical formulations and the results will be discussed in a separate report.

If the results of the verification are positive, further steps may consist of:

- o drafting pre-designs primarily checking whether and which structural solutions are possible
- o checking the cost effectiveness
- o investigation of fundamentals including optimizations using models (2D and scale)
- o field experiments.

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PART C

MORPHOLOGICAL RESPONSE TO MEASURES AND STRATEGIES

PART C: MORPHOLOGICAL RESPONSE TO MEASURES AND STRATEGIES

6 MORPHOLOGICAL RESPONSE TO MEASURES

In Chapter 5 a number of promising measures have been preliminarily selected. The selection was based upon a qualitative assessment of the effectivity viz. the hydraulical and morphological response of the river to these measures. This response is elaborated hereafter more quantitatively (see also Section 5.6) along the lines indicated in the scope of study (Section 2.5).

Method of erosion control	Preselected measures	Study components		
		Response of flow at bifurcation	Response of channels	Response of outer bank
Redistribution of flow at bifurcation	Permeable groynes, impermeable groynes, surface vanes	6.1	6.2	6.4
Redistribution of flow in cross-section	Sills, bottom vanes or surface vanes	-	6.3	6.4

Table 6-1: Study Components

As the objective of the measure is to control the erosion of the outer bank, the characteristics of the relation between the measures (e.g. main dimensions) and the erosion process have to be studied. This is done in steps as indicated in Table 6-1, and in Fig. 6-1.

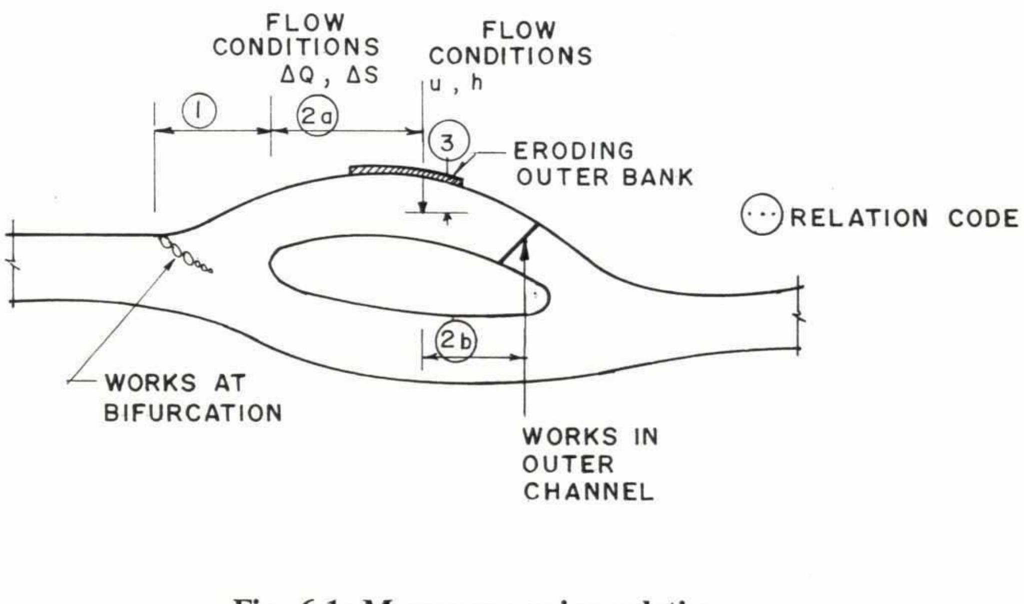

Fig. 6-1: Measures-erosion relation

The figure shows that the sought relation between measures and the erosion of the outer bank can be split up into three relations:

1. works at bifurcation versus flow conditions at the channel entrance
2a. flow conditions at the channel entrance versus flow conditions at the toe of the outer bank
2b. works in the outer channel versus flow conditions at the toe of the outer bank
3. flow conditions at the toe of the outer bank versus the erosion of the outer bank.

In the following sections a computational framework (sometimes called a simplified model) is outlined to establish these relations.

6.1 RESPONSES AT BIFURCATION

The works at the bifurcation, permeable groynes or surface vanes, aim at redistributing the flow there. So a computational method is sought to establish the relation between the main characteristics of the works and their impact on the flow distribution.

6.1.1 Set-up of a Local Model

For the set-up of a local mathematical model of the works at the bifurcation the following was considered :

The works

1. As a bandal is basically a surface vane attached to a pile screen (which is a long permeable screen) the mathematical formulation of the bandal is sought with a variable vane draft. If the draft becomes zero, the bandal becomes a needle screen. Thus the characteristics of surface vanes and permeable groynes are combined.
2. The angle of attack should be a variable. In other words the orientation of the bandal with regard to the flow direction may vary.
3. The length of the bandal should be variable as the bandal may cover either the full width of the channel entrance or only a part of it.
4. The density of the vertical piles and there diameter cause a blocking factor, which should be a variable.

The channels

5. The three channel branches at the bifurcation are schematized to have rectangular cross-profiles with variable widths.
6. The river bed at the bifurcation is supposed to be flat.
7. The angles between the axis of the channels are not variable; in fact the following schematization is used (see Fig. 6.1-1).

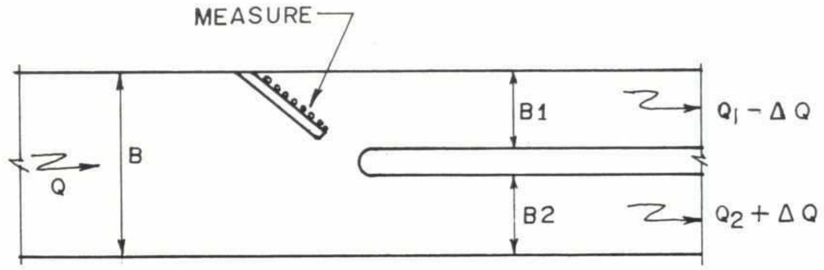

Fig. 6.1-1: Schematization of bifurcation

The water flow

8. The inflow (Q) into the model area is supposed to be constant, only steady conditions ($\delta/\delta t = 0$) are considered.
9. The initial conditions per channel are uniform ($\delta/\delta x = 0$), which means that without the bandal the water slopes in the three branches are equal to the bed slopes. The velocities in the channels may be different.
10. For the situation with the bandal, the flow field needs to be assessed and schematized. The bandal causes additional resistance in the entrance of the outer channel resulting in a reduction of flow there. Also the deflection of the surface flow due to the angle of the bandal screen is a factor to be considered.

The computations may comprise the following components:

- o the Bernoulli equation and the continuity equation are applied between a cross-section upstream and downstream of the bandal
- o the increase of the water level upstream of the bandal is limited by an approximative condition. If the water level increases furthermore discharge is deflected by the screen.
  This limitation is based on bend flow upstream of the groyne and an assumed area with more or less stagnant water
- o downstream of the bandal a water flow with or without a drowned hydraulic jump will be considered
- o the forces on the screen due to resulting water level differences are calculated
- o the resistance of the piles is estimated.

The sediment flow

11. Initially (without the bandal) the uniform flow will carry an equilibrium concentration of sediments. The transports in the channels will be assessed using a transport formula for suspended sediments.
12. For the situation with the bandal the transport distribution is changing (ΔS) as a consequence of ΔQ but also as a consequence of the shape of the sediment concentration vertical. A relative higher portion of the transport will pass under the bandal screen.
13. If possible also the scouring under the bandal will be taken into account.

6.1.2 Modelling

Considering the aspects to be covered in the local model, described above, it is concluded that no existing one-dimensional package can be used. A special first order mathematical formulation needs to be elaborated.

Thereafter programming can be done using a spreadsheet program in e.g. LOTUS 1-2-3

6.1.3 Running the Model

The main I/O parameters are :

Input :
- o channel dimensions : B, B_1, B_2, bed slope
- o channel roughness : Chézy value
- o screen dimensions, length, draft, orientation
- o pile dimensions, diameter, inter distance, number
- o water flow Q, Q_1, Q_2
- o sediment flow characteristics

Output :
- o redistribution of flow : ΔQ, ΔS
- o backwater curve
- o drowned water jump characteristics
- o forces on bandal.

6.1.4 Verification

A real overall calibration and verification of the model is impossible as data on this type of works are scarce and not systematic. This holds both for field data and model data.

A global verification can be done, comparing part of the model results with similar parameters such as:

- o the head over the screen
- o the flow velocities under the screen
- o effects of a screen on the flow pattern
- o hydraulic load on screen and piles.

6.2 RESPONSE IN CHANNELS TO WORKS AT BIFURCATION

The effects of the bandal on the flow (the ΔQ and ΔS values) determined with the local model of the bifurcation are the upstream boundary conditions of the channels. The response of the channels to the ΔQ and ΔS values is to be computed. In other words : what are the hydraulic (water levels) and morphological (bed-levels) consequences in the channels to ΔQ and ΔS ?

These responses can be determined in two steps :

- o response per individual channel
- o response of the combined channels.

6.2.1 Response per Channel

As a first step the response of each channel is assessed separately, using a first order theoretical approach.

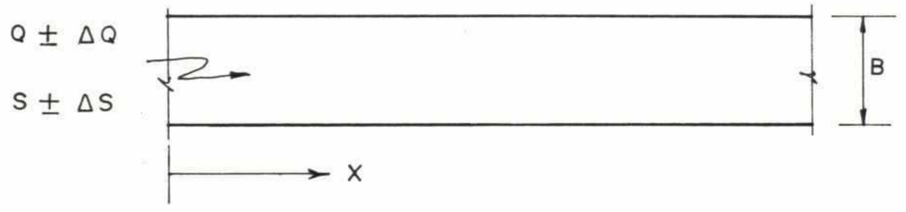

Fig. 6.2-1: Response per channel

The output gives the water and bed levels as a function of time. Thus an estimate is obtained to what extent and how fast hydraulic and morphological changes can be realised as a function of ΔQ and ΔS. The main advantage of these computations is the gaining of insight into the relation between the relevant parameters and the sensitivity thereof.

6.2.2 System Response

The channels need to be coupled as the changing of the hydraulic and morphological conditions in one channel affect the flow distribution at the bifurcation and consequently the conditions in the other channel. Therefore 1D modelling can be applied with probably a new version of MIKE 11. An example of the system schematisation is given in Fig. 6.2-2.

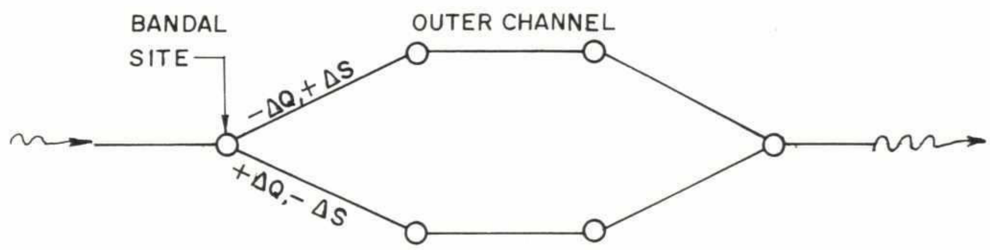

Fig. 6.2-2: System schematization

In this way the results of the local model of the bifurcation and the computations of the responses per channel are combined. The main output consists obviously of the flow velocity and the remaining water depth in the outer channel (u,h).

6.3 RESPONSE TO WORKS IN OUTER CHANNEL

The preliminary selected works in the outer channel consist of:

- o sills downstream of the eroding outer bank
- o bottom vanes or surface vanes near the toe of the eroding outer bank.

The computational set-up to establish the relation between the works in the outer channel vs the flow conditions (u,h) at the toe of the outer bank are outlined in the following sections.

6.3.1 Sills

The main characteristic of the sill is its relative height h_s/h_o. The objective of the computation is to find $u,h = f(h_s/h_o, t, \ldots)$. Some characteristic details have been presented in Fig. 6.3-1.

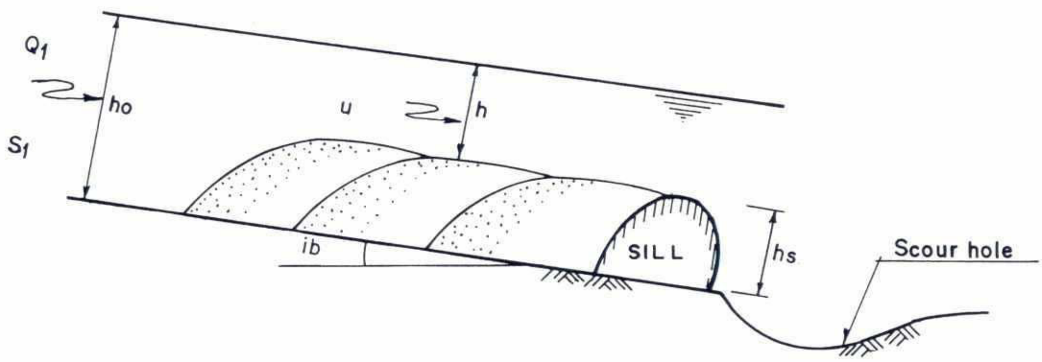

Fig. 6.3-1: Sedimentation upstream of sill

Two approaches will be followed:

- o schematization in a MIKE 11 model with several branches,
- o analytical solutions for a schematized situation.

In a MIKE 11 model a sill can be schematized as an overflow weir if the sill is made of stable materials or as a local rise of the river bed, if the sill is made by dredging for example. Both schematizations will be investigated.

The local scour downstream of the sill can probably be neglected, because it has less influence on the flow field than the sedimentation upstream of the sill.

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6.3.2 Bottom Vanes

To establish the relation between the main characteristics of the bottom vane and the flow velocities and water depth at the toe of the outer bank a local computational model is to be made.

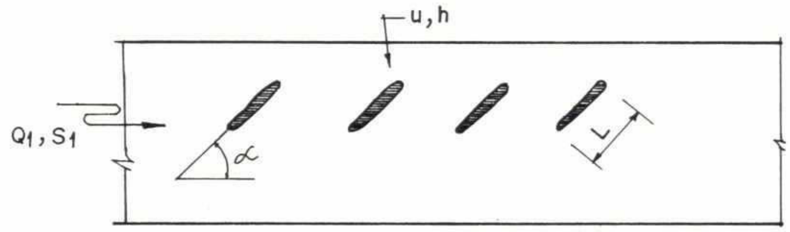

Fig. 6.3-2: Bottom panels in local model

The model should give as the main output:

$u,h = f(H_p, L, \alpha, \ldots\ldots)$

in which

H_p = height of the panel
L = length of the panel
α = angle of attack.

The model will have strong similarities with the model of the single bandal as described in Section 6.1. A complicating factor is the number of panels and the spacing.

6.3.3 Surface Vanes

The effects of surface vanes in a row before the migrating outer bank on the flow and water depth need also to be described in a purpose made local mathematical model.

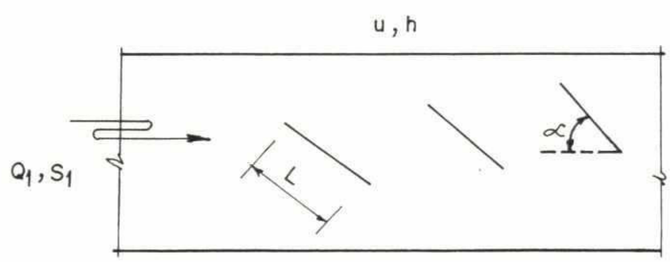

Fig. 6.3-3: Surface vanes in local model

With the aid of the model the relation is sought of

$$u,h \;=\; f\,(H_d,\, L,\, \alpha\,)$$

in which

H_d	=	draft of the screen
L	=	length of the screen
α	=	angle of attack.

The model will have strong similarities with the model set-up described in Section 6.1 on the effects of a single bandal. The complication here is also the repetition of the screens, the number and the interdistance.

6.4 REDUCTION OF BANK EROSION

6.4.1 General

The last shackle in the chain is to relate the effects of the works at an upstream confluence, resulting in sedimentation in the outer channel and subsequent reduction of flow, to the erosion of the outer bank. For this it is required to establish a relation between the conditions of the outer channel (see previous section) and the bank erosion.

Several options are available for assessing the reduction in bank erosion rates, varying from detailed 2-dimensional modelling of flow and sediment transport, bed topography and bank erosion to more straightforward approaches relating e.g. overall stream power to bank erosion rates. Here a summary of two of the possible options is given and the more simple one is adopted within the frame-work of the present pre-feasibility study. If needed in a later stage always a more detailed model, requiring also much more field data, can be used. The description of the two possibilities is preceded by some general information on bank erosion processes.

6.4.2 Bank Erosion Processes

River bank erosion is a complex phenomenon in which many factors play a role. The rate of bank erosion is determined by flow, sediment transport, bank properties and (sometimes) water quality (Mosselman, 1992). These different aspects are discussed hereafter shortly.

The flow in eroding outer bends is affected by the channel characteristics like discharge, dimensions and slope. In bends also the bend characteristics, in particular the radius of the bend is important, together with the overall planform of the river. In the bend a helical flow pattern is generated that is of major importance for the resulting bed topography. The sediment transport in bends is determined by the flow characteristics and the characteristics of the bed material, and, if relevant, the characteristics of the bank material if that is going to be a substantial part of the bed material.

The bank properties include bank material weight and texture, shear strength and cohesive strength, physico-chemical properties, bank heights and cross-sectional shape, groundwater levels and permeability, stratigraphy, tension cracks, vegetation and constructions. Many of these factors are only approximately known, hence often one has to lean heavily on field observations for calibration of models to be used. The influence of water quality is assumed to be of minor importance for the rivers in Bangladesh.

A distinction can be made between bank erosion of non-chesive bank material and cohesive banks. Non-cohesive soils are eroded by the peeling off of individual particles (Thorne & Osman, 1988). Cohesive banks erode by mass failure during discrete events when a critical stability condition is exceeded. The banks of the Jamuna river are fairly steep and consist of material with apparently some cohesive characteristics. This seems to be substantiated by the sampling of the bank material recently carried out within the frame-work of FAP1: the bank consist of about 80% fine sand and 20% silt/clay responsible for the cohesive behaviour. The material disintegrates very quickly when disturbed. The subsoil investigations carried out in the FAP 21 project test areas (with the exception of Nakalia) only showed some clay contents in the upper most layers. Hence it is assumed that the steep to nearly vertical banks are mainly due to "apparent cohesion" of basically non cohesive (silty) fine sands. Apparent cohesion is caused by the surface tension of the pore water. This would explain the fact that the bank material disintegrates that quickly when disturbed and that it has hardly any resistance to the eroding force of the flow.

For cohesive soils a possible approach would be to model each mass failure separately. A more commonly used alternative (particularly true for the Jamuna) is to consider the time averaged bank migration that can be modelled as a direct response to the erosion of the toe of the bank. This erosion of the toe of the bank is determined by the fluvial entrainment of material and can be divided into:

- o lateral fluvial entrainment of bank material, and
- o near bank degradation of the river bed.

The lateral entrainment is a function of:

- o erodibility coefficient,
- o actual shear stress, or actual flow velocity
- o critical shear stress or critical flow velocity.

The actual shear stress on a bank is about 0.75 times the longitudinal bed shear stress in the channel. Near bank degradation is a function of:

- o erosion depth of the river bed
- o bank slope angle.

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Bank erosion has a cyclic behaviour: the river bed erosion results in steeper slopes of the bank and an increase in bank height. After collapse of the upper part of the bank, debris will be accumulated at the toe of the bank. The river flow has to remove this debris before the erosion of the bank can continue. The amount of debris is a function of the height of the banks. Therefore the bank retreat is also a function of:

- o bank erodibility
- o total bank height
- o critical bank height.

The total bank height is the sum of the freeboard of the bank above the water level and the near-bank water depth.

In the above the bank erosion model the following phenomena are not taken into account:

- o bank erosion by wind waves and by ship waves
- o bank erosion by groundwater flow.

For more details on these aspects reference is made to e.g. DHV (1990). Nevertheless, note that especially the erosion by wind waves during so-called Nor'westers probably can be substantial, at least at certain locations. Its overall effect can be included during any calibration using actual field data on bank erosion. In the Jamuna River strong turbulent boils are observed during the monsoon period, which may also have an effect on the bank erosion. For the time being it is assumed that this effect is included in existing sediment transport equations.

6.4.3 Available Models

A number of models is available for predicting bank erosion along river bends. Here two models are summarized, notably the model of Mosselman (1992) and the method proposed by Hickin and Nanson (1984).

The bank erosion model of Mosselman (1992) is restricted to the erosion of cohesive banks by river flow. It computes the bank erosion using the time averaged approach briefly outlined in the previous Section. For a sloping bank of cohesive materials the bank retreat can be calculated by:

$$\frac{\delta n_B}{\delta t} = E\,\frac{(u_w^2 - u_{wcr}^2)}{u_{wcr}^2} + G.\frac{h_w - h_{wcr}}{h_{wcr} + H_{fb}} - \frac{1}{\tan\varphi}.\frac{\delta z_b}{\delta t} \qquad (6.4\text{-}1)$$

in which $\delta n_B / \delta t$ = the bank retreat as a function of time (m/s),E = erodibility coefficient related to shear stresses, G = erodibility coefficient related to the bank height, u_w = flow velocity near the bank (m/s), u_{wcr} = critical flow velocity (m/s), h_w = water depth near the bank (m), h_{wcr} = critical water depth near the bank, if the water depth is smaller than the critical water depth no bank erosion occurs (m/s), H_{fb} = free board of the bank above the water level (m), φ = bank slope, z_b = bed level, t = time.

The above equation shows clearly that the application of this equation requires that insight is available into the velocity and water depth in the outer bend and the slope of the bed in longitudinal direction. Mosselman (1992) obtains these data from a detailed 2-dimensional mathematical model that simulates flow in bends (inclusive the helical flow and its effect on the main flow) and the sediment transport (bed load!) and that predicts the bed topography. For the present study the use of such a model is not appropriate, hence approximative methods should be used.

A method which requires less detailed data is the method proposed by Hickin and Nanson (1984). They relate the bank erosion to:

- stream power of the river
- bend characteristics
- resistance of the bank.

For the stream power of the river the following definition is used:

$$\Omega = \rho g Q i \qquad (N/s) \qquad (6.4\text{-}2)$$

with

ρ	= density of water	(kg/m^3)
g	= acceleration of gravity	(m/s^2)
Q	= river discharge	(m^3/s)
i	= river slope	(-)

The resistance of the bank is represented by a coefficient Y_b, that is shown to be a function of the particle size composition of the bank material, conform Fig. 6.4-1.

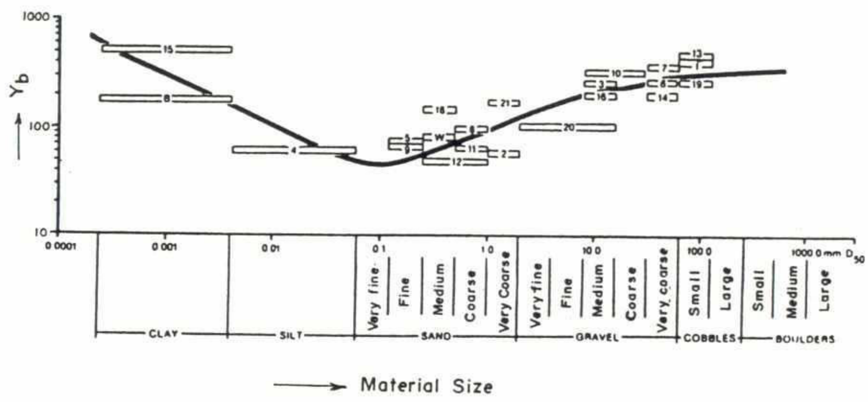


Fig. 6.4-1: Relation between coefficient of resistance to lateral erosion and bank material size

(Source: Hickin & Nanson, 1984)

Hickin & Nanson (1984) demonstrate on the basis of a number of rivers (as indicated by individual figures in the graph presented in Fig. 6.4-1) in Canada that the bank erosion rate can be computed with the following relations:

$$E = \frac{2.5}{(R/B)} M_{2.5} \quad (for \ \frac{R}{B} > 2.5)$$
$$E = \frac{(R/B-1)}{1.5} M_{2.5} \quad (for \ \frac{R}{B} < 2.5) \tag{6.4-3}$$

where E = annual erosion (m), $M_{2.5}$ = annual erosion for R/B = 2.5 (-), R= radius of curvature of the bend (m), B = width of the river (m). The value of $M_{2.5}$ is found from the following relation:

$$M_{2.5} = \frac{\Omega}{Y_b H} \tag{6.4-4}$$

where H = outer bank height (m). Hence what is needed for this method is i, Q, R, B and H. Furthermore the value of Y_b has to be verified possibly by comparing with field data.

The reduction of the erosion as a function of R/B was found from the field data used by Hickin & Nanson (see Fig. 6.4-2 A-C). The reason for the maximum at R/B = 2.5 is that for this relative curvature the velocities along the bank are largest. A further decrease leads to the main flow going through the inner bend, thus reducing the outer bend scour. Klaassen & Masselink (1992) analysed data from the Jamuna river and found a similar shape of the bank erosion rates (see Fig. 6.4-2 D).

The following observations are made regarding the method of Nanson & Hickin:

(1) The method can easily be applied: after calibration the only parameter not readily available is H, but methods to estimate this value are available (see hereafter). For the application to the Jamuna river with very large bank erosion rates the effect of bank erosion products on the outer bend depth should be taken into account (Mosselman, 1989).

(2) For the discharge Q, Hickin & Nanson (1984) propose the five year flood discharge which is slightly larger than the bankfull discharge. Here it is assumed that the bankfull discharge can be taken as well.

(3) It is amazing that the duration of the flood is not included in the method. During the calibration of the method this should get extra attention.

Recently the method of Hickin & Nanson (1984) was tested versus the Jamuna bank erosion data presented in Klaassen & Masselink (1992). It was found that the method, when using the value of Y_b as indicated for sand with a diameter of 0.2 mm, yields already fair results.

The method of Hickin & Nanson satisfies the condition that it should be readily applicable within the present pre-feasibility investigations, and is in this respect preferable over methods like Mosselman (1992). It is therefore proposed to use the method of Hickin & Nanson (1984) after calibration to the Jamuna conditions. As can be seen from Equation (6.3), the reduction of the discharge in a channel results in a reduction of the bank erosion rates.

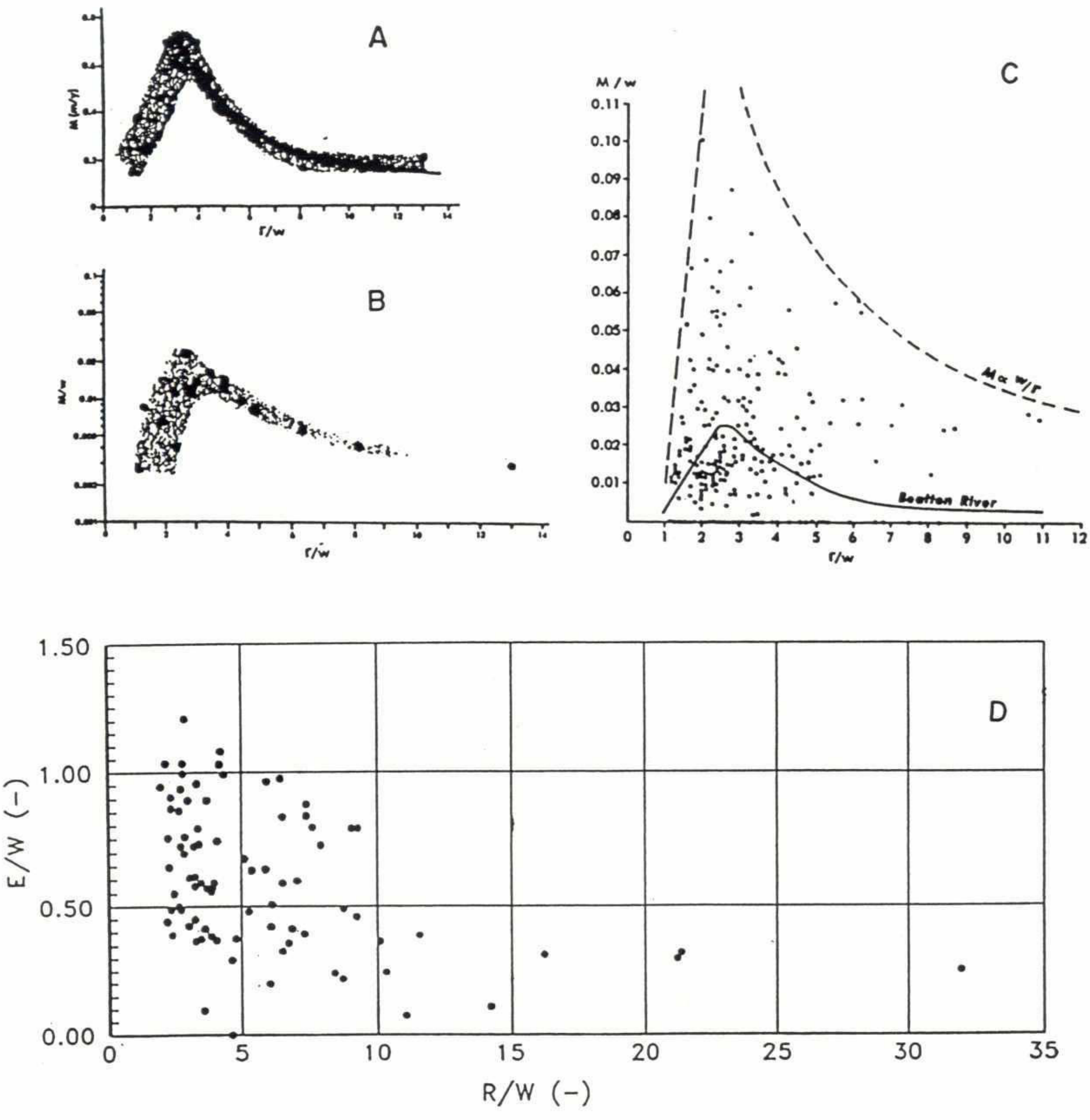


Fig. 6.4-2: Relation of bend migration E(A) and relative bend migration E/B (B, C and D) and relative curvature R/B for (A and B) bends in Beaton river, Canada, (C) many rivers in Canada (Nanson & Hickin, 1984) and (D) Jamuna river, Bangladesh

(Source: Klaassen & Masselink, 1992)

The value of H, which is needed for the application of the method of Hickin & Nanson, can be determined approximately from the following equation:

$$H = \bar{h} + \Delta h_B - \Delta h_{erosion\ products} \qquad (6.4\text{-}5)$$

where Δh_B = increase in water depth due to bend scour (m), and $\Delta h_{erosion\ products}$ is the subsequent reduction due to the bank erosion products. The value of Δh_B can be determined in an approximative way from a graph that is given as Fig. 6.4-3. It appears that Δh_B is a function of $\hat{h}$, B/R, A (a secondary flow coefficient) and θ (mean Shields parameter). Reference is made to Mesbahi (1992) for specific details to determine Δh_B.

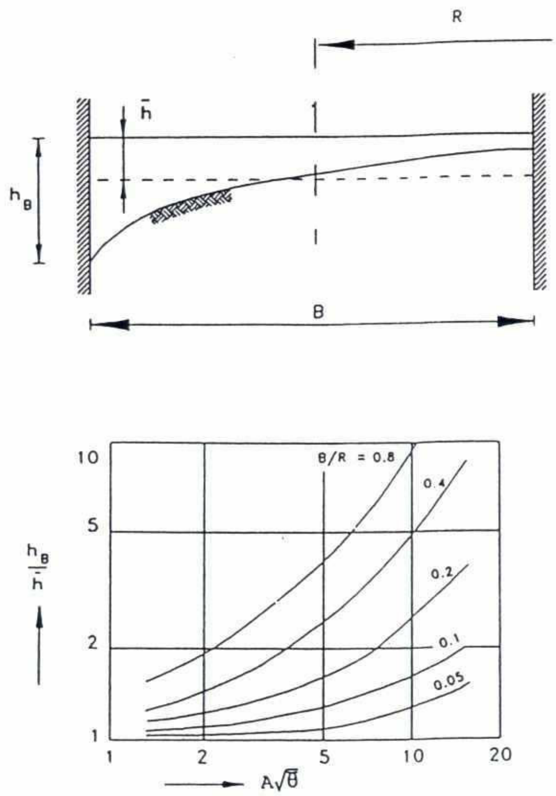


Fig. 6.4-3: Maximum bend scour

(Source: Mesbahi, 1992)

For the conditions of the Jamuna River the value of $\Delta h_{erosion\ products}$ can be derived from an equation proposed by Mosselman (1989).

6.4.4 Calibration and Verification

To apply the method of Hickin & Nanson (1984) calibration will be needed. This can be done comparing the results of the application of the method to field data from the Jamuna River. The latter have been and are being obtained during the analysis of satellite images within the present study. It is proposed to use only a part of the data for calibration. The other data can then be used for verification purposes.

7 RESPONSE OF A RIVER SYSTEM TO AFPM

7.1 GENERAL

One of the important aspects when considering the application of AFPM for the Jamuna River is the estimation of the response of the river system. This response can be divided into:
(i) response at short notice, and
(ii) the long term consequences for the river characteristics.
In this respect also a differentiation in space can be made. Responses at short notice usually are limited in distance at which their effect is noticeable, while long term consequences will in due time become apparent over the whole distance of the river.

Regarding the **response at short notice** again a difference can be made, notably the effect that is actually the purpose of the actual measure, and non-intended side effects. Regarding the former effect of e.g. placing of bandals, it is of course noticeable in the branch which has to be closed: gradually sedimentation will occur, the discharge in the channel will be reduced and also the bank erosion will gradually decrease in time. Non-intended side-effects can be noticeable in the other branches that are subject to less sediment: degradation and widening will occur, the discharge in the channels will increase and gradually also the bank erosion along curved reaches will increase. Understanding of this response is important for avoiding not-acceptable backlashes. Some initial ideas for assessing this response quantitavily have been presented in the preceding chapter.

The **long term response** of the river to AFPM may be that the river characteristics may change. How serious this will be depends fully on the extent of the AFPM measures. If only one or two channels are closed yearly, the overall impact will be very small. If, however, the strategy of the AFPM measures is to reduce the total width of the river to say 10 km and to tackle all channels that are tending to cross an imaginary line 5 km on both sides of the centerline of the river, then the measures to be taken are much more . In that case it may be expected that also the response of the river to this strategy will be much more serious. This may lead to a reduction of the braiding index (the number of channels per cross-section) and this in time may lead to larger channels, with deeper scour holes and even larger bank erosion rates.

Identification and assessing the extent of these responses are very important as these responses determine to a large extent socio-economic benefits and damages due to AFPM. Methods to predict the responses of the river system to AFPM measures are discussed in this chapter. The discussion presented here is only a summary of a more extensive literature investigation to be reported upon later in a separate ANNEX. Based on the literature survey and a more detailed analysis of the characteristics of the Jamuna River, a method for application will be selected. In later stages of the project, when a better insight has been obtained in the AFPM strategies, the impact of these different strategies will be evaluated using the selected method(s) for assessing the river's response. Finally at the end of the project the applicability of these methods to other rivers in Bangladesh will be assessed within the framework of a study into the applicability of AFPM techniques to other rivers.

In Section 7.2 the imposed and the dependent variables in a river system are dealt with and discussed. In Section 7.3 a summary is given of prediction methods for especially channel width, sinuosity, number of channels, and total width of the river system. In Section 7.4 an overview is given of activities to be undertaken in the near future to select the most appropriate prediction techniques (if there is a choice) for the Jamuna River type of conditions.

7.2 IMPOSED AND DEPENDENT VARIABLES

7.2.1 General

A river is a complicated system in which quite a number of variables are present. This holds especially for a braided river system like the Jamuna River. Over the last century the understanding of the interrelationship between the different variables in a river system has gradually increased, but even at present this understanding is still quite limited for braided sand bed river. Predictions can only be made for very schematised conditions.

One of the more common assumptions is that one channel-forming discharge can be identified, that is responsible for "shaping" the river bed. Usually the bankfull discharge is selected for this channel forming discharge. In the case of the Jamuna River this is even more complicated, because there is a difference between the "bar full" discharge (about 38,000 m^3/s, see BRTS 2nd Interim Report) and the "bank full" discharge starts to inundate the flood plain (about 44,000 m^3/s, see Klaassen & Vermeer, 1988).

A second problem is the fact that the conditions in a river are often very much time-dependent. This holds especially for a braided sand bed river (Klaassen & Vermeer, 1988) for some examples). The changes in the number of channels, width and depth of the individual channels and the total width of the channels are so quick that the momentary conditions can differ greatly from the "average" conditions. Still it is assumed here that average conditions can be defined, and the present analysis deals especially with these average conditions. In addition here it is assumed that on the average the river system is in equilibrium.

7.2.2 Variables and Equations

Assuming that one channel-shaping discharge (here referred to as the dominant discharge Q_d) has been selected (to beals with), the following parameters can be identified in a braided river system: the dominant discharge Q_d, the sediment transports the river has to carry, the characteristic size of the bed material D, the valley slope i_v, the number of channels n, the total width taken by the river, the sinuosity p of the individual channels, the slope i of the river, the bankfull width B_b of a channel, the bankfull depth h_b, and the velocity u_b (during bankfull conditions), the roughness coefficient (do) and the actual width, depth and velocity of the river. In addition here the hydraulic radius R_b is introduced as a variable for reasons that will become clear later.

It is not completely clear which are the imposed and which are the dependent variables. As is shown in ANNEX 2 of this report, this depends on the time scale being considered. If only the conditions at the time scale of a flood are considered the channel characteristics can be considered as imposed. If the time scale considered is the time scale of the morphological processes (typically between 10 and 100 years), then the channel dimensions are very much dependent on the geology of the catchment, the climate and the river training carried out. The variables B, h and u are not relevant in this case any more, as they follow from the channel characteristics. Hence the imposed variables on a morphological time scale are Q_d, S, D, i_v and the dependent variables are n, B_t, p, i, B_b, h_b, R_b, u_b and C (in total 9). See also Fig. 7.2-1, where the main parameters in a river system are schematically indicated. Hence for a free flowing (and "shaping") river system nine equations are needed to find solutions for all parameters. For rivers very much subjected to river training works the number of variables may be different as the river training works may have fixed some variables (like number of channels, sinuosity, width, etc.). For a quickly reacting river like the Jamuna River the time scale of the morphological processes and the engineering time scale are in the same order of magnitude.

The number of equations needed exceeds the number of equations available. In fact only six equations are available, that are in principle undisputed although their actual formulation may not be known fully. These are:

(1) continuity equation $Q = Bhu$ (undisputed)

(2) momentum equation which in its simplest form (steady uniform conditions) takes the form of the Chézy equation or the Manning equation (undisputed),

(3) roughness predictor (only approximately known),

(4) sediment transport predictor (only approximately known),

(5) definition of sinuosity $p = i_v/i$ (by definition, hence undisputed),

(6) relation between the hydraulic radius and the width, depth and some other parameters (only approximately known).

Consequently, additional equations are needed for solving this set of equations. For solving all dependent variables in fact three additional equations are missing. Suggestions for these additional equations are discussed in Section 7.3.

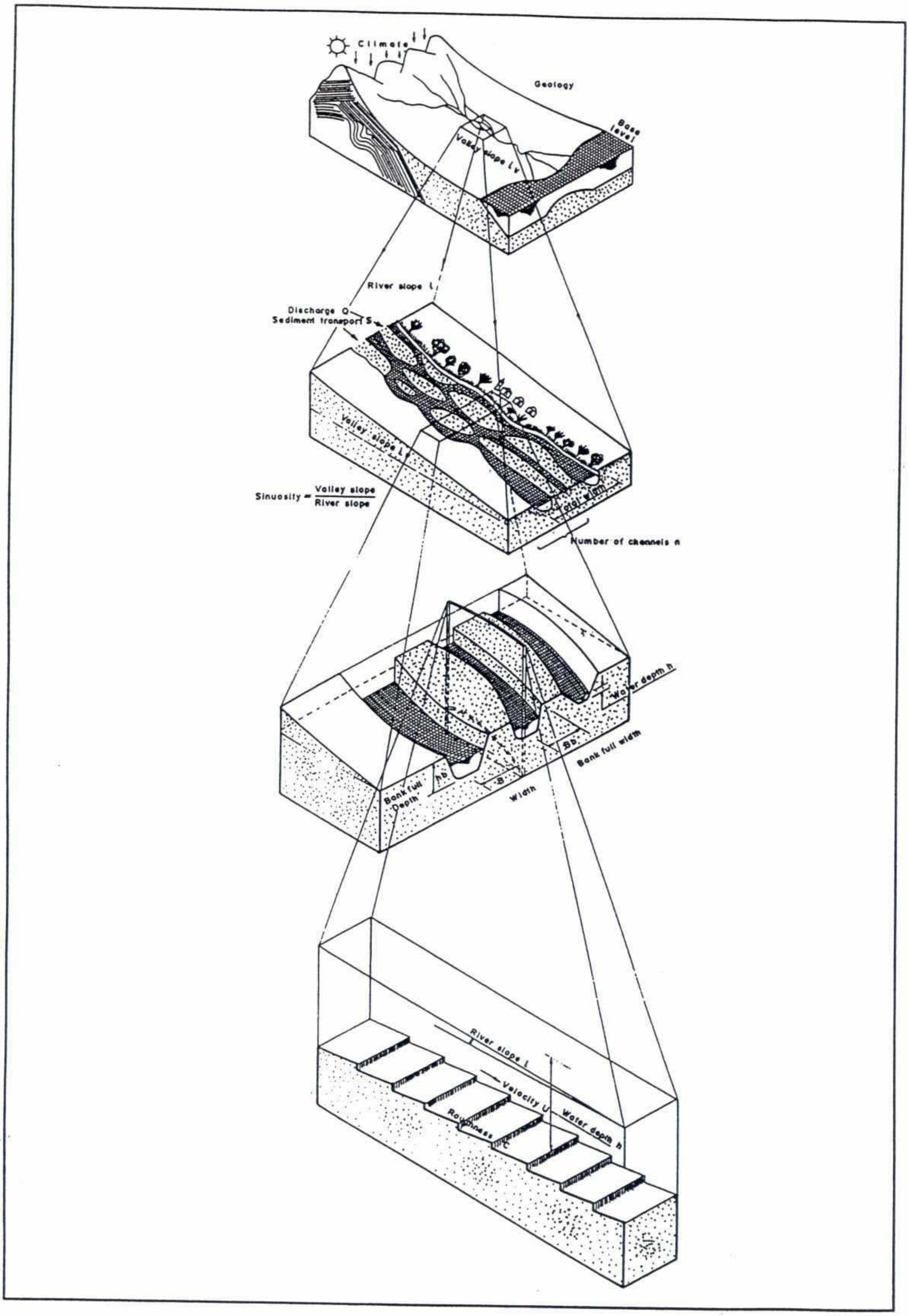


Fig. 7.2-1: Main parameters in a river system

7.2.3 Dependent Variables and AFPM

As already briefly indicated above, river training works may reduce the number of dependent variables by imposing one or more parameters. Here it will be indicated what are the possibilities in this respect as far as AFPM is concerned. It is obvious that whether or not dependent variables are fixed depends on the strategy adopted.

Within the frame-work of the FAP 22 studies strategies for AFPM have not yet been detailed, but for the present purpose it is required to identify some of the options. Some possible options are the following:

Option 1 Close only very aggressive channels, probably some 2 or 3 per year.

Option 2 Try to reduce the total width of the braided belt by consequently closing the most outward channels, and by forcing artificial cutoffs.

Option 3 Construct gradually more bank protection works at vulnerable places, which will lead to a narrower channel (conform what has been the case downstream of Sirajganj over the last decades).

Option 4 Try to create a transition of the braided river with multiple channels to a meandering river with one channel.

Option 5 As option 4 but in combination with river training works to stabilize the alignment of the river.

In terms of dependent variables these different options can be described as:

(1) reducing the number of braids marginally (option 1) via substantially (option 2) to extreme (option 4 and 5),

(2) reducing the total width of the braiding belt (option 2 and 3),

(3) fixing the sinuosity of the single channel left (option 5).

It can therefore be concluded that the prediction of the response of the river to AFPM measures should comprise the response of the river to reducing its total width, and its number of braids, and to fixing its sinuosity.

In the following Section the available prediction techniques for the dependent variables in a natural river system are summarized. In a further step possibilities to assess the response of a river system to AFPM measures are reviewed in Section 7.4.

7.3 PREDICTION METHODS FOR NATURAL RIVERS

7.3.1 General

As is shown in Section 7.2, for a river system the number of dependent variables (9) exceeds the number of undisputed and approximately equations available (6). Hence additional equations are needed. These additional equations, in combination with the undisputed equations presented above, can be used to derive prediction methods for the response of a river system to changes owing to river training works like studied within the framework of FAP 22. In the present Section 7.3 prediction methods for natural rivers are considered. In the subsequent Section 7.4 it is indicated how the impact of river training works contemplated within the framework of FAP 22 can be evaluated using the same prediction methods, and to what extent calibration will be needed and how that should be done.

Until now only river systems were considered here. There is however no major difference between a natural river and a canal excavated in alluvial soils that is operating with some sediment transport. Hence in the following canals in regime (that do on the average not exhibit erosion or sedimentation) are discussed first. These canals are the most simple systems to consider because the number of channels n is 1 and the sinuosity p is also 1. The only parameter "missing" is a predictor for the width. Next more complicated systems are considered, starting with a meandering river (still with $n = 1$, but $p > 1$) and finally a braided river system (where $n > 1$ and also $p > 1$). Ultimately methods for the prediction of the total width of a river system are discussed. In the following discussion two types of additional equations are distinguished: (i) empirical relations, and (ii) theoretical relations.

7.3.2 Width Predictors

7.3.2.1 General

The prediction of the width of a stable channel has been an issue for many decades, however mainly for irrigation canals newly to be excavated. A stable channel in this respect is a canal in alluvium in which on the average neither scour of the canal's banks and bed nor deposition takes place. Here on the average should be underlined as many canals that are classified as stable go through periods of deposition but this is followed by periods of scour. Often this is a yearly cycle related to the variation in sediment content of the water taken in from canals.

Hereafter a difference is made between empirical predictors and theoretical predictors.

7.3.2.2 Empirical Predictors

Empirical formulae for the desirable width for sand bed canals to be stable are available since the beginning of this century. They were developed on the Indian subcontinent for the design of large irrigation systems in the Punjab. Not only a predictor for the canal width was given.

The regime theory as it is often referred to (although there is no theory behind its development) presents three equations respectively for the width, the depth and the slope of the canal. As an example here the equations originally proposed by Lacey (1929) are quoted, using the original notation in imperial units:

$$P = 2.67\ Q^{\frac{1}{2}} \tag{7.3-1}$$

$$R_h = 0.473\ Q^{\frac{1}{3}} f^{-\frac{1}{3}} \tag{7.3-2}$$

$$i = \frac{1}{1750} f^{\frac{5}{3}} Q^{-\frac{1}{6}} \tag{7.3-3}$$

where P = wetted perimeter (ft) Q = design discharge (ft^3/s), R_h = hydraulic radius (ft), and f = silt factor introduced by Lacey and to be determined from the following equation:

$$f = 1.76 \sqrt{D_{50}} \tag{7.3-4}$$

where D_{50} = bed material size (in the formula of Lacey in **mm!**). The above equations were derived for sand bed canals with fairly cohesive banks, bed material size in the range of 0.1 to 0.5 mm and low sediment concentrations (100 to 2,000 ppm). Because they are based on empiry, application in areas outside the Indian sub-continent should be done with care. Recently Stevens & Nordin (1985) have given an interesting review of the basis of the regime equations of Lacey (1929), indicating the relation between the regime equations and nowadays generally accepted laws (see the previous section) and underlining the weak theoretical background of this regime approach.

Simons & Albertson (1960) derived a more comprehensive set of equations including more data form India and Pakistan and data form the USA. It was found that a differentiation can be made as to the bank and bed material, the widest and most shallow canals corresponding to conditions with "sandy bed and banks".

In a further development of regime equations attempts have been made to derive regime equations for natural rivers as well. The problem in applying this approach to natural rivers is, however, that natural rivers tend to have quite a variation in discharge while canals often carry most of the time the design discharge. A commonly made assumption is that the bankfull discharge of a river is also the discharge doing most of the work, and hence can be taken as the basis for regime equations. Sometimes also a flood with a certain frequency (1.5 or 2 year flood) is used. As examples of regime equations for rivers here the regime equations derived by Hey and Thorne (1983) and others for gravel bed rivers and Klaassen & Vermeer (1988) for braided sand-bed rivers are presented.

For gravel bed rivers Hey and Thorne (1983) derived regime equations based on data from the UK. The equations for the width can be written as:

$$B = C_1\ Q^{0.5} \tag{7.3-5}$$

where B = bankfull width (m), Q_b = bankfull discharge (m^3/s), and C_1 = coefficient depending on the vegetation on the banks, according to the following index:

Coefficient C_1	Bank vegetation
4.33	Grass banks, no trees
3.33	1-5 % covered with trees and shrubs
2.73	5-50 % covered with trees and shrubs
2.33	> 50 % covered with trees and shrubs

What is clear form this table is that for the rivers considered here the vegetation has an important effect on the regime width.

Also for the bankfull depth and the slope of gravel bed rivers relations are proposed by different authors. As an example here the relation for the slope proposed by Bray (1973) is given:

$$i = 0.059\ Q_2^{-0.333}\ D_{50}^{0.586} \tag{7.3-6}$$

where Q_2 = 2 year flood (m^3/s).

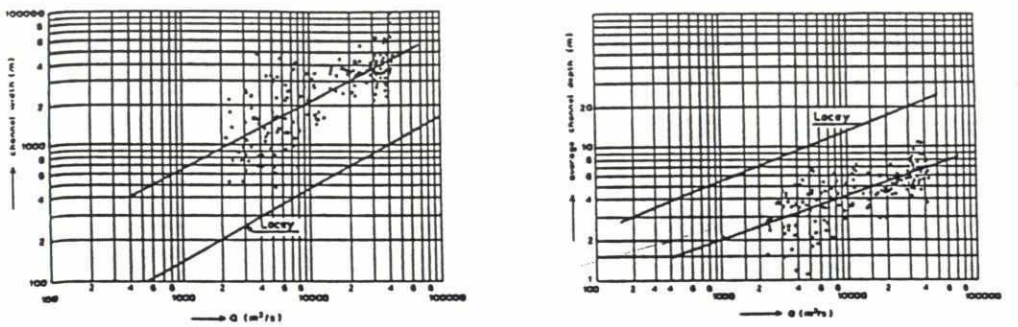
Relations for braided sand bed rivers are given by Klaassen & Vermeer (1988). In fact these equations were derived from an analysis of cross-sections from the Jamuna River, and some spurious correlation was introduced, because the bankfull discharge was divided over the channels according to their conveyance. Because here very wide channels are considered the hydraulic water depth can be substituted by the bankful water depth, while the width is substituted for the wetted perimeter.The derived equations are presented hereafter:

$$\overline{h_b} = 0.23\ Q^{0.32} \tag{7.3-7}$$

$$B_b = 16.1\ Q^{0.53} \tag{7.3-8}$$

where B_b = bankfull width (m) and h_b = bankfull depth (m).

This is also shown in Fig. 7.3-1, where a comparison is made between the regime equations derived by Klaassen & Vermeer and those proposed by Lacey (1929). It is observed that the Jamuna channels are much wider and more shallow than the Punjabi canals for which the Lacey equations were derived.



(a) width versus discharge (b) average depth versus discharge

Fig. 7.3-1: Regime equations for Jamuna river channels compared with the Lacey (1929) regime equations

(Source: Klaassen & Vermeer, 1988)

The following remarks are made considering the regime equations discussed above:

(1) There is a fair correspondence as far as the powers in the equations is concerned. The width of both canals and rivers scales approximately with $Q^{1/2}$.

(2) Apparently there is a substantial influence of the vegetation for the rivers considered by Hey & Thorne (1982). Considering the wide channels in the Jamuna River it may be assumed that for that river, however, the influence of the vegetation is negligible. This is in line with the observation that the bank erosion along the Jamuna River is not different for vegetated banks compared to unvegetated banks (Klaassen & Masselink, 1992).

(3) The slope of regime canals and also of rivers are inversely related to the dominant discharge: the larger the discharge, the smaller the slope and vice versa. The implication of this for the occurrence of braided systems will become clear later.

7.3.2.3 Theoretical Width Predictors

In the previous subsection empirical predictors were discussed. There have however also been attempts to predict the width of a canal and of a river using a more theoretical approach. These attempts can be classified as:

(1) Stable channel approach

(2) Lateral exchange approach

(3) Extremal hypotheses.

Although, as will be shown later, only the latter approach has resulted in usefull results for the present study also the other two are discussed here briefly.

Re (1) Tractive force approach (stable channel approach)

This approach assumes that the channel carries little or no sediment transport. It may therefore be assumed that for all particles, both in the bed and in the banks, the critical conditions are not exceeded. The critical shear stress is of course a function of the particle size (according to Shields (1936) and of the cohesion). Furthermore the effect of the slope of the banks on the stability has to be accounted for. Finally also the lateral variation of the shearstress in a channel has to be taken into account. The maximum shear stress on the banks for a trapezoidal cross-section with fairly steep slopes, is usually only about 75 % of the shear stress on the bed. Taking these factors into account it is possible to derive theoretical cross-sectional shapes that satisfy stable conditions. Related approaches have been followed by Thorne (1982), relating the channel stability to the stability of the banks, and Singh (1983), relating the shear stress (reduced according to the reasoning above) on the banks to the limiting maximum shear stress. As shown by Bettess et al. (1987) these approaches lead to results that are not compatible with the empirical relations discussed before. Furthermore these approaches cannot be used for rivers.

Re (2) Lateral exchange approach

An interesting approach was followed by Parker (1978a and 1978b). He argued that equilibrium in sediment transporting channels is achieved if there is a balance between opposing mechanisms causing erosion and deposition. For the banks of stable sand-silt rivers he argued that there should be an equilibrium between (i) erosion of the banks due to gravity affected lateral bed load from the banks towards the bed, and (ii) deposition on the banks due to the lateral diffusion of suspended material generated by the non-uniform distribution of suspended sediment across the width. Using this approach he developed a regime equation for the depth of a regime channel which reads as:

$$\frac{h}{D} = 85.1 \left[\frac{w_s}{\left(\frac{\rho_s}{\rho} - 1\right) g D^{1/2}}\right] i^{-1/2} \tag{7.3-9}$$

where ρ_s and ρ are the densities of sediment and water (kg/m^3) respectively, and w_s = settling velocity of the suspended sediment (m/s). Also for this expression Bettess and al (1988) demonstrate that it leads to results that are not compatible with empirical relations.

Re (3) Extremal hypotheses

Over the last decade or so width predictors based on extremal hypotheses have been developed that seem to result in fairly good predictions of the width (and hence the depth and slope) of canals and rivers. The following extremal hypotheses have been proposed:

(1) Minimum stream power (Chang, 1980);
(2) Minimum unit stream power (Yang and Song, 1979);
(3) Maximum sediment transport capacity (Ramette, 1979 and 1990; White et al, 1982);
(4) Minimum energy dissipation rate (Yang et al, 1981);
(5) Maximum friction factor (Davies and Sutherland, 1980).

Not all these hypotheses are discussed here extensively. To illustrate the concept of an extremal hypothesis the definition as stated by Chang(1980b) is given here: "For an alluvial channel the necessary and sufficient condition of equilibrium occurs when the stream power per unit channel length, ρgQi, is a minimum subject to given constraints. Hence, an alluvial channel with water discharge Q, and sediment (discharge) S as independent variables, tends to establish its width (B), depth (h) and slope (i) such that ρgQi is a minimum. Since Q is a given parameter, minimum ρgQS also means minimum channel slope S."

The assumption by White et al (1982) reads: "....for a particular water discharge and slope the width of the channel adjusts to maximise the sediment transport rate." A similar hypothesis was proposed by Ramette (1979).

To illustrate the use of the method of White et al here an example is presented of a channel with a discharge of 500 m^3/s and a sediment size of 40 mm. Assuming that the slope of the channel is $2.14*10^{-3}$, computations of the sediment transport were carried out for different assumed widths. This was done using the roughness predictor of White et al (1980) to predict the water depth and the sediment transport predictor of Ackers & White to predict the sediment transport. The result is given in Fig. 7.3-2, where the computed sediment transport is plotted versus the assumed width. It is clear that a maximum occurs for B = 43 m, where the sediment transport corresponds to 100 ppm.

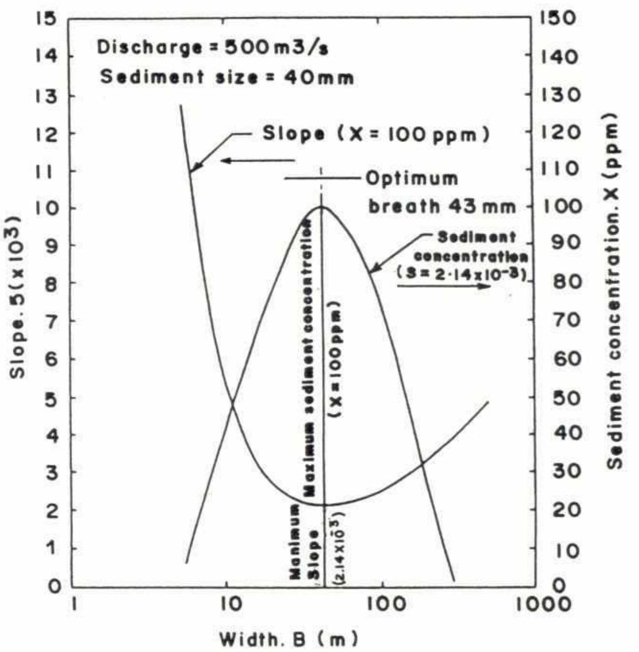


Fig. 7.3-2: Example of variation of slope and sediment concentration as a function of width

(Source: White et al, 1982)

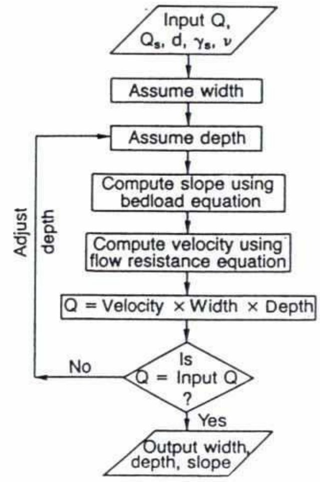


Fig. 7.3-3: Flow chart showing major steps in computation of Chang (1979)

In the same Fig. 7.3-2 the result of an analysis keeping the sediment transport constant (to 100 ppm) and again varying the width is presented. In this case, corresponding to the minimum power hypothesis of Chang (1979a), a minimum slope is found for again a width of 43 m corresponding to a slope of $2.14*10^{-3}$. Hence this example shows that in this case a minimum exists and that the minimum power hypothesis yields the same extremal as the maximum sediment transport hypothesis. This was also shown in a more general way by White et al (1982).

In Fig. 7.3-3 the procedure followed for computing the minimum slope according to the method of Chang (1979) is indicated. In the figure the computation is indicated for one channel width. This has to be repeated for a series of widths, and in a final step the width has to be selected that yields the lowest slope.

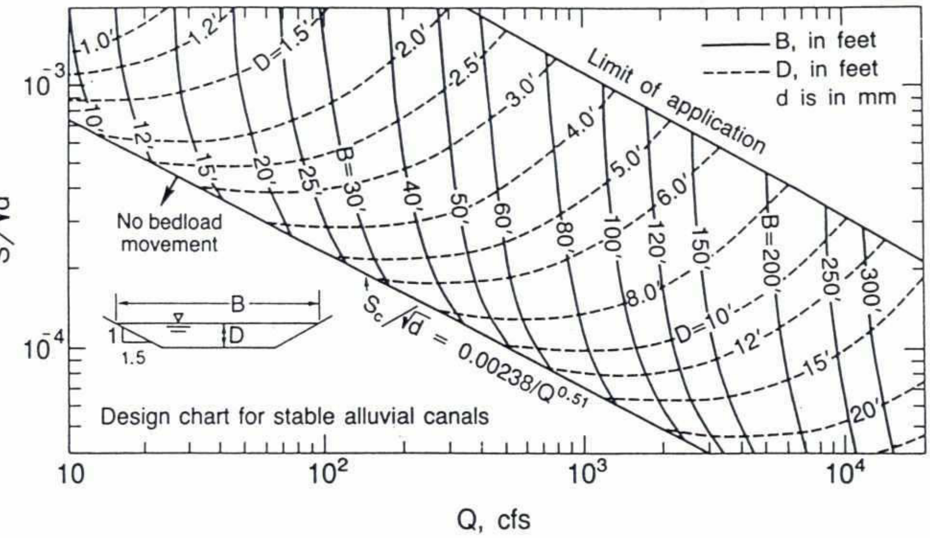
As is indicated in the figure, the method needs of course the selection of a sediment transport predictor and a hydraulic roughness predictor. For the method of White et al (1982) and for all the other methods also such predictors have to be selected. Table 7.3-1 present an overview of the predictors included in the various models proposed. Furthermore it is indicated which extremal hypotheses is used. In addition all methods include the hydraulic roughness being used instead of the water depth. This is of course logic because predictions were also made for narrow trapezoidal channels. The method used by White et al (1982) assumed that the side slope z (z horizontal and 1 vertical) was given by Smith's (1974) relationship:

$$z = 0.5 \qquad if \quad Q < 1 \; m^3 / s \tag{7.3-10}$$

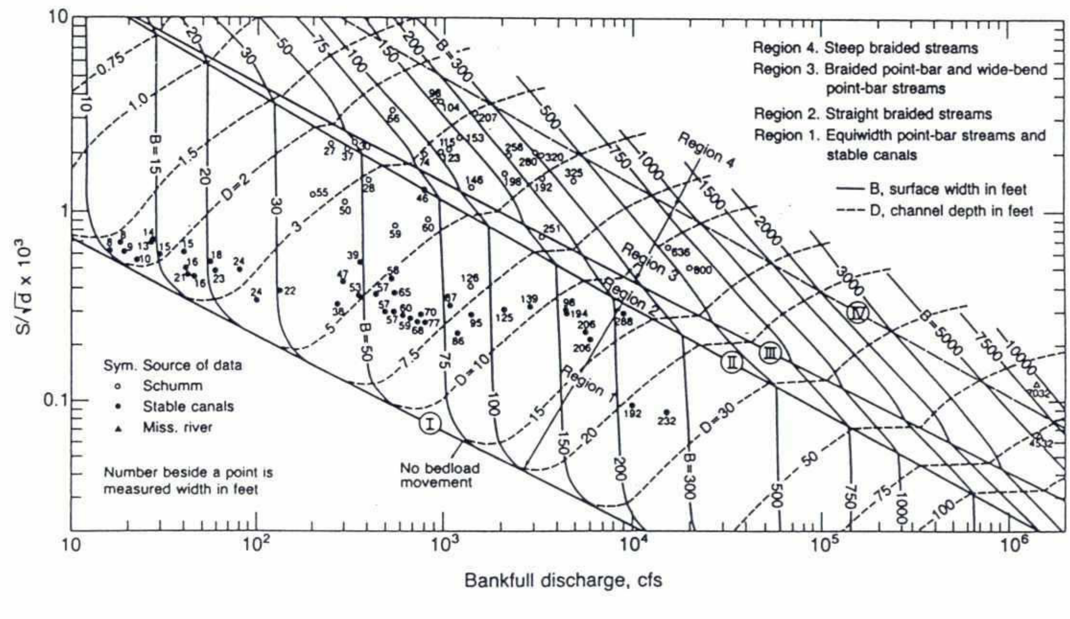
$$z = 0.5 \; Q^{\frac{1}{4}} \qquad if \quad Q > 1 \; m^3 / s \tag{7.3-11}$$

Predictor	Equation	Prediction method: Chang (1979; 1980 and more recent)			Prediction method: Yang (1979 and more recent)	Prediction method: Ramette (1979; 1990)	Prediction method: White et al (1982)
		Stable canals	Gravel bed rivers	Sand bed rivers			
Transport predictor	DuBoys	x		x			
	Einstein & Brown			x			
	Engelund & Hansen (1967)						
	Ackers & White (1973)						x
	Parker / Chang (1980)		x				
Roughness predictor	Lacey	x					
	Meyer-Peter & Müller					x	
	Engelund & Hansen (1967)			x		x	
	White et al (1980)						x
	Bray (1979)		x				
Extremal hypothesis	Minimum stream power	x	x	x			
	Minimum unit stream power				x		
	Max energy dissipation						
	Max sediment transport					x	x
	Max friction factor						
	Max Froude number					x	
Bank roughness	Hydraulic radius	x	x	x		x	x
	Water depth						

Table 7.3-1: Comparison of different extremal hypotheses proposed



(a) Stable channels



(b) Sand-bed channels

Fig. 7.3-4: Design graphs developed by Chang (1979, 1980)

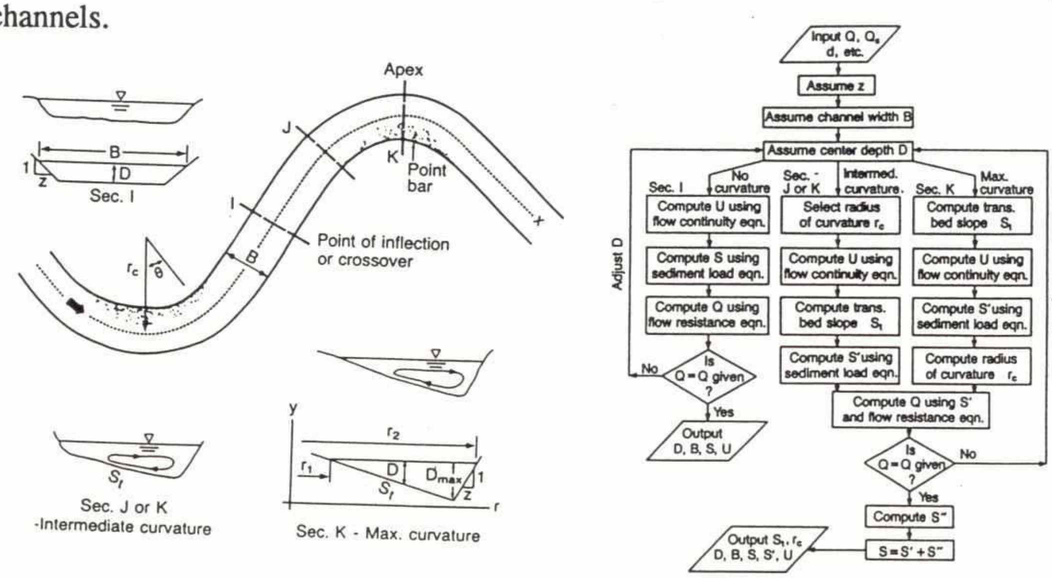
Different researchers have elaborated the use of the extremal hypothesis up to different levels:

(1) Chang (1987) has developed design charts for stable alluvial channels which are dependent on the accepted side slope (see Fig. 7.3-4a). The width in these charts corresponds to the surface width during bankfull stages. In a further analysis Chang (1980b) developed similar graphs for gravel-bed rivers and sand bed rivers:

   (a) Gravel-bed rivers are assumed to have low bed load transport only, hence the hydraulic roughness is determined only by the grain roughness.

   (b) Sandbed rivers were studied by including the effect of meanders, arriving at design graphs for sand bed rivers. As is shown in Fig. 7.3-5 Chang considered three characteristic sections in a meandering river and analysed the minimum stream power concept for each of them separately. It was found that the predicted width for the three sections did hardly differ. The result is a design graph for sand-bed rivers that is presented here as Fig. 7.3-4b. The graphs cover a range of bankfull discharges from 10 to 2,000,000 ft^3/s, corresponding to a range of 30 l/s to 57,000 m^3/s. For the Jamuna river the bankfull discharge of separate channels varies between 2,000 and 44,000 m^3/s so in principle the chart can be used.

(2) White et al (1982) have compared their results with field and flume data. As is shown in Fig. 7.3-6, a fair agreement was obtained. Furthermore they have elaborated their results by preparing a book with tables from which the dimensions of a straight channel can be obtained. This book of course is limited by the use of the two predictors mentioned above. Only for channels satisfying these predictors it can be expected that a good prediction is made. The book covers channels with bankfull discharges up to 1,000 m^3/s, which is definitely not sufficient for the Jamuna channels.



(a) Cross-sections considered **(b) Flow-chart computation**

Fig. 7.3-5: Extremal hypothesis by Chang (1980) applied to meandering sand-bed rivers

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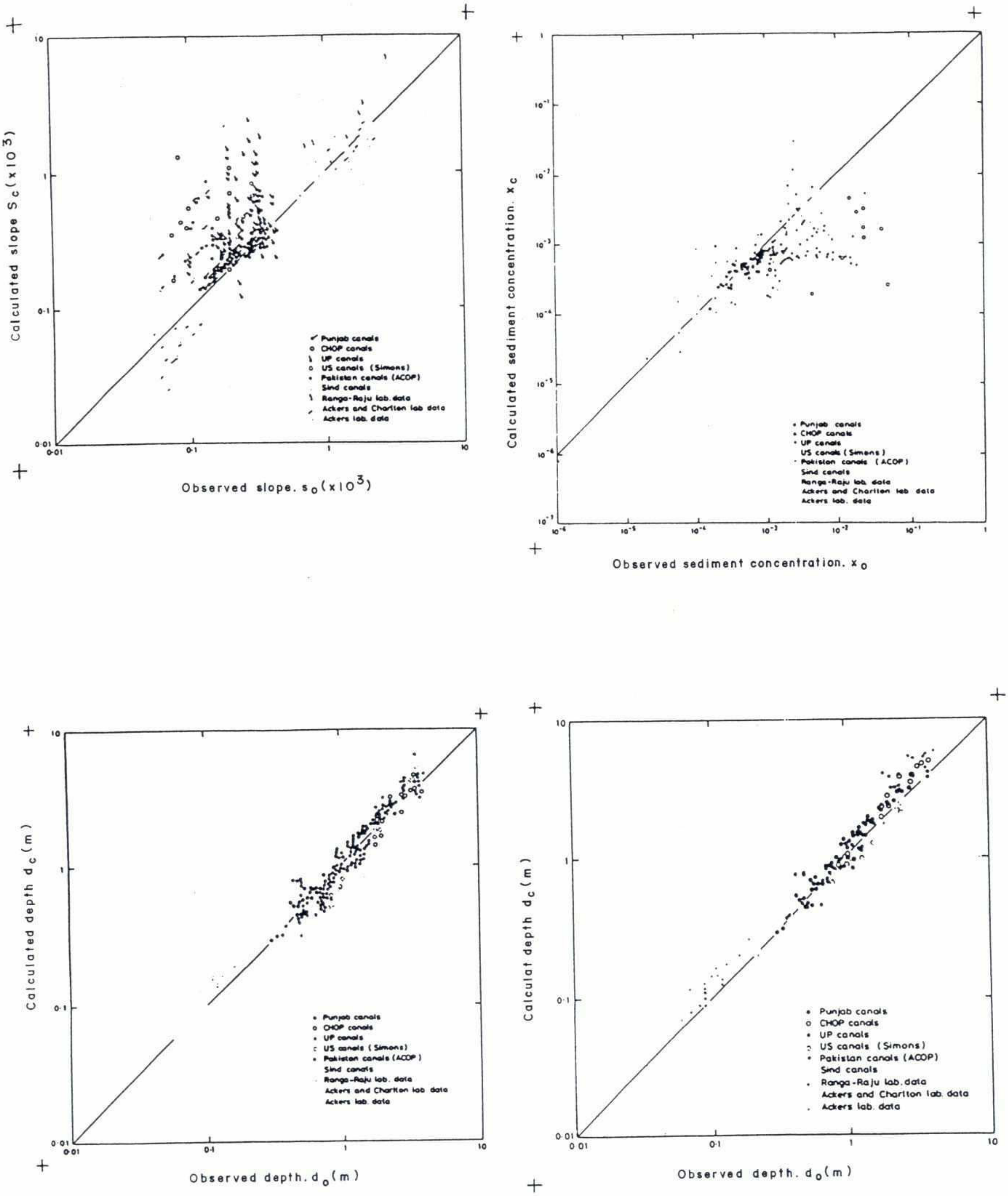

Fig. 7.3-6: Comparison of maximum sediment transport method of White et al with (mostly) field data

(Source: White et al, 1982)

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(3) Ramette (1990) had developed analytical expressions for the regime equations derived by him based on the assumption of maximum sediment transport and maximum Froude number, which read as:

$$R = 9.5 * 10^{-2}\ Q^{0.233}\ D^{0.26}\ i^{-0.41} \tag{7.3-12}$$

$$B = 8.4\ Q^{0.527}\ D^{0.006}\ i^{-0.077} \tag{7.3-13}$$

As is shown by Ramette (1990) these equations compare quite favourably with the empirical equations derived by Klaassen & Vermeer (1988). Introducing $i = 6*10^{-5}$ and $D = 0.18*10^{-3}$ m yields equations that are almost similar to the equations (7.3-7) and (7.3-8) yields:

$$R = 0.54\ Q^{0.233} \tag{7.3-14}$$

$$B = 16.9\ Q^{0.527} \tag{7.3-15}$$

which are amazingly similar to the empirical relations.

The following remarks are made regarding the width predictors described above:

(1) In an article by Griffiths (1984), in which the five extremal hypotheses described above are reviewed critically it is stated that "... the hypotheses .... are incompatible with conventional sediment transport and flow resistance equations." Furthermore it is stated that "The hypotheses in their present form are unacceptable." Chang (1984) in a reply stated that in his analysis "Griffiths ignored the effect of channel bank slopes and shear stress reduction near the banks that are so important in the width formation of alluvial streams ...". It may be doubted whether in the end these extremal hypotheses are usefull for Jamuna type of rivers where the aspect ratio (channel width divided by the channel depth during bankfull conditions) is in the order of 100 and hence the influence of the banks vanishes. This should be investigated in a later stage of this project (see also Section 7.4).

(2) The above predictors have been derived for straight channels. When applying these methods to rivers the effect of meandering should be included. Only Chang (1980) has developed a method for this.

7.3.3 Predictors for the Sinuosity

For natural rivers also the prediction of the sinuosity is important. Assuming that the valley slope is given, the prediction of the sinuosity of a meandering channel can be obtained in a straightforward way once the slope of the river has been established. By definition the sinuosity p is given by:

$$p = \frac{i_v}{i} \tag{7.3-16}$$

where i_v = valley slope (-), and i = slope of the river (-). The slope of the river follows in a straightforward way from the prediction of the width because once the discharge Q, the sediment transport S, the particle size D and the width B are fixed, the velocity, the hydraulic roughness and the slope become dependent variables.

In principle there are three possibilities, depending on the relative values of i_v and i:

(1) $i = i_v$:
The slope of the river corresponds to the slope of the valley: the river will remain straight.

(2) $i > i_v$:
The slope of the river is larger than the valley slope. The only way the river can cope with this situation is by flooding the floodplain, causing sedimentation in the river channel. In due time this will also lead to an increase of the valley slope due to sedimentation in the floodplain, but as indicated in Section 7.2.2 this will only take place at a time scale much larger than the morphological time scale of the river.

(3) $i < i_v$:
The slope of the river is smaller than the valley slope. The river can cope with this condition by starting to meander. Due to the meandering the length of the river between two points will increase and hence the slope along the river will decrease until the slope is reached which corresponds to its regime width.

Regarding the latter possibility there is of course a limit to the sinuosity. Typical sinuosities in nature vary between 1.5 and 2.5. If the difference between valley slope and river slope is too much then the river has to cope with this situation in another way. As will be explained in Section 7.3.4 one way of doing this is to increase the number of channels, hence to start braiding. For an explanation on the above see Fig. 7.3-7.

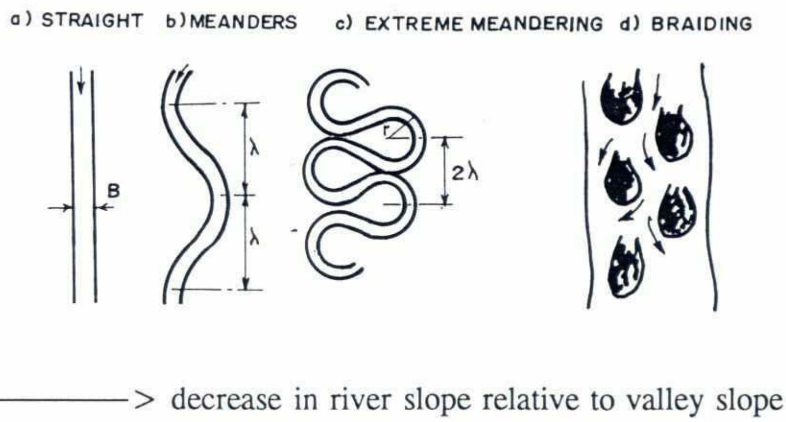

Fig. 7.3-7: Meandering as a way to cope with a difference between valley slope and river slope

(Source: Ramette, 1990)

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7.3.4 Predictors for the Number of Channels

7.3.4.1 General

An important parameter for the characterization of a river is the number of channels in a cross-section. First of all, it indicates whether a river is meandering (one channel) or braided (two or more channels). Secondly, it represents the braiding intensity. Methods to predict the number of channels per cross-section are discussed here. This Section deals with the prediction of the number of channels of a river system as a function of the independent variables. In this respect a distinction can be made between the transition from one channel to more channels (usually associated with the transition from meandering to braiding) and the occurrence of numerous channels. The transition from meandering to braiding has been studied by many researchers, the number of studies on the number of channels as a function of the independent variables Q, S, D and i_v is very limited. The present section deals with these different aspects, whereby both empirical and theoretical studies are dealt with.

7.3.4.2 Transition from Meandering to Braiding

(a) Empirical methods

Early attempts to determine the conditions for the occurrence of either meandering or braided rivers resulted in empirical classification graphs, the first one by Leopold and Wolman (1957), who plot bankfull discharge against channel slope. From that, they derive an equation for the separatrix between meandering and braiding

$$i = 0.0116\ Q^{-0.44} \tag{7.3-17}$$

where Q is the bankfull discharge in m^3/s and i is the channel slope. If the actual channel slope is steeper than i, the river will be braided, whereas a milder slope will lead to a meandering river. Later studies provide similar separation criteria, but comparisons with data are not very satisfactory (Bettess and White, 1983). Ferguson (1984) proposes to include the bed material size of the alluvial channels as additional parameter to improve the predictions.

(b) Theoretical methods

More recently more theoretical predictions for the classification of river planform have been developed. A recent example is the study by Struiksma and Klaassen (1988), who base a tentative criterion for the transition from a meandering to a braided river on the theoretical and experimental work of Struiksma et al (1985). They argue that the transition starts when the damping length of steady alternate bars becomes negative, so that bars grow exponentially in downstream direction. The key parameter for the threshold between meandering and braiding is then the interaction parameter, λ_s/λ_w. This is a ratio of two adaptation lengths, viz.

$$\lambda_w = \frac{C^2}{2g}\ h_0 \tag{7.3-18}$$

for water motion, and

$$\lambda_s = \frac{1}{m^2\ \pi^2}\ \frac{B^2}{h_0^2}\ f\ (\theta_0)\ h_0 \tag{7.3-19}$$

for the deformation of the bed. The symbols denote B = river width (m), C = Chézy coefficient for hydraulic roughness,

$f(\theta_0)$ cross-sectional average of function for the influence of gravity pull along a transverse bed slope

g acceleration due to gravity

h_0 cross-sectional average of water depth

m transverse mode, indicating the number of channels per cross-section

π 3.14159..

Note that

$$\lambda_s(m) = \frac{1}{m^2}\lambda_s\ (1) \tag{7.3-20}$$

Struiksma and Klaassen (1988) show that their criterion is compatible with the earlier empirical classification graphs and Ferguson's (1984) inclusion of sediment size.

Also Chang (1981, 1987) proposes a method for determining the transition from meandering to braiding.

7.3.4.3 Prediction of Number of Channels in a Braided System

There are hardly any methods to predict the number of channels in a braided river system. Some qualitative arguments have been given by White et al (1983) and Struiksma & Klaassen (1988). The reasoning Bettess & White (1983) is the following: If i_v greatly exceeds i (e.g. determined with one of the methods described in Section 7.3.2), then the river has two possible ways of coping with this situation. One is to start to meander, hence to increase its sinuosity. There is however a limit to this. The maximum observed sinuosity is about 2.5. The alternative way is to split up into two or more branches. According to what was explained in Section 7.3.2 (see e.g. Equation 7.3-3), a channel that carries a smaller discharge has a larger slope than a channel with a larger discharge. Hence by doubling, tripling, etc. a river can also cope with a difference between i_v and i. According to White & Bettess (1983), a river system has a preference for a maximum number of channels because in this way the stream power (which is equivalent to Q.i) is maximised. Fig. 7.3-8 illustrates the response of a river to increasing valley slopes according to Bettess & White (1983).

Struiksma & Klaassen (1988) use essentially the same reasoning: if a channel becomes unstable a possible reaction of the river system is to increase its number of channels thus reducing the ratio the interaction parameter λ_s/λ_w.

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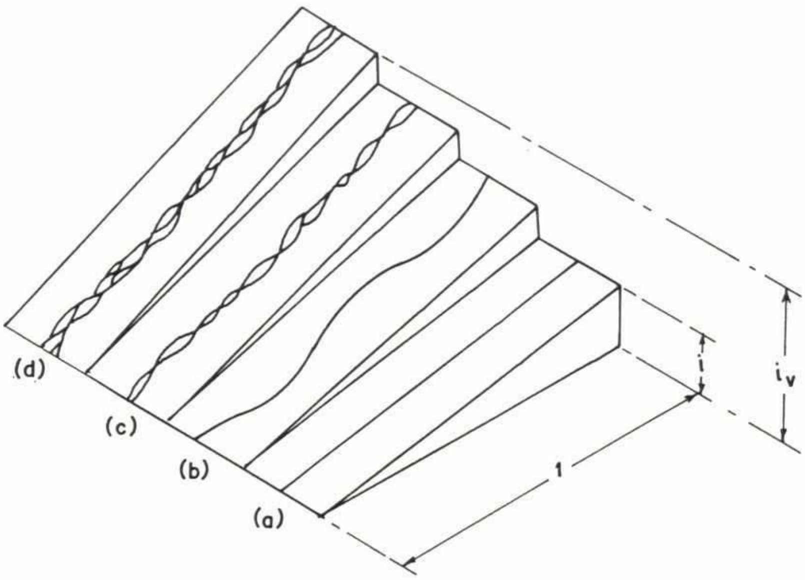

Fig. 7.3-8: Channel pattern for increasing valley slope

(Source: Bettess & White, 1983)

Hereafter prediction methods for the number of channels in a braided river system are described briefly. A difference is made between:
(a) empirical relations, and
(b) theoretical predictions.

Re (a) Empirical relations

Only one empirical relation was found in the literature, notably the relation proposed by Vincent et al. This relation, based on an analysis of field data, has been tentatively applied to the Jamuna River, (done within the framework of the Jamuna Bridge Project). However this approach did not yield encouraging results.

Re (b) Theoretical prediction

Two theoretical prediction methods were identified. One is the method by Bettess & White (1983), already discussed briefly above. The method consists of assuming a number of channels (1,2,3, ...) and dividing the discharge and the sediment transport over these channels. Then, using the method outlined in Section 7.3.2.3, the resulting slope of the channels is determined. The (maximum) number of channels is selected that yield a slope only slightly less or equal to the valley slope. If the slope is slightly less than the valley slope, then it is assumed that the river slope will cope with that by having slightly sinuous braids.

The method of Bettess & White (1983) is essentially a "channel approach", because it considers the channel as independent items. A slightly different approach is an approach which can be described as a "bar approach", developed at DELFT HYDRAULICS.

Here the number of braids is found by analysing the coexistence and competition of several bar modes in transverse direction. In this approach it is assumed that due to instabilities periodic disturbances develop which depending on the conditions can be alternate bars, islands, multiple islands, etc. This can be studied by evaluating the marginal stability curve, given by

$$\frac{m^2}{2}\,\frac{\lambda_w}{\lambda_\xi\,(1)}\,\frac{1}{(\,k\,\lambda_w\,)^2} + \xi = \frac{(\,2 + X\,)\,(\,1 + bX\,) + (\,1 + X\,)\,(\,b - 3 - bX\,)}{4\,(\,k\,\lambda_w\,)^2\,(\,1 + X\,)^2 + (\,2 + X\,)^2} \qquad (7.3\text{-}21)$$

in which b = exponent of power-law sediment transport formula, k = streamwise wave number, ξ = coefficient for the effect of streamwise bed slopes on sediment transport, and X denotes

$$X = \frac{kB}{m\pi} \qquad (7.3\text{-}22)$$

Again, the interaction parameter appears as one of the main parameters. The equation for the marginal stability curve can be rewritten in the general form.

$$\frac{\lambda\,(1)}{\lambda_w} = F_m\,(\,\frac{kB}{\pi}\,) \qquad (7.3\text{-}23)$$

in which F_m depends on m. An m^{th} mode is linearly unstable if the marginal stability curve has a minimum below the actual value of $\lambda_s(1)/\lambda_w$. If modes up to $m = m_{max}$ can be linearly unstable, the number of channels per cross-section could be expected to be equal to m_{max}.

The equation for the marginal stability curve stems from a linear analysis, so that its formal validity is still restricted to infinitely small deviations from a plane bed. It may well be that non-linear interactions change the number of channels. An analysis by Schielen et al (1992), for instance, reveals that more channels might appear during further evolution of the bars. On the other hand, the number of main channels in the Jamuna is much smaller than the theoretical value of m_{max}, but in addition there are minor channels as well. The Jamuna consists of a system of channel hierarchies with dominant first-order channels, smaller second-order channels and even smaller third-order channels (Williams and Rust, 1969). We assume that the number of main channels, m_*, is a function of the number denoting the highest mode that is linearly unstable.

$$m_* = f\,(\,m\,) \qquad (7.3\text{-}24)$$

Further research is needed to the establishment of this functional relationship. It should be noted, however, that apart from non-linear interactions and a selection of first-order channels, also other factors may cause a difference between m_* and m.

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The non-uniformity of the envelope banks may force certain patterns in the river, and the emergence of bars above the water level due to discharge variations may have an influence as well.

7.3.5 Predictors for the Total Width of the River

No predictors for the width of a braided river system were found during the literature search reviewed here. Ramette (1983) proposes a method to determine the width occupied by a meandering river, and empirical relation are presented in Leopold et al (1964), which can be generalized to a form:

$$A = C_1 \, Q^{C_2} \qquad (7.3\text{-}25)$$

where A is the amplitude of the meander belt and the coefficients C_1 and C_2 vary between 2.7 and 18.7 and 0.99 to 1.2, respectively. It is difficult to visualize how these method could be applied to a braided river system.

7.4 SELECTION OF PREDICTION METHOD

7.4.1 Present Status of Prediction Methods

In the above Sections a review is given of state-of-the-art of prediction methods for channel characteristics of braided river systems with fine sand as bed and bank material. In particular the prediction of the width of the channels, the sinuosity, the number of braids and the total width of the river system are dealt with. Summarizing it can be stated that for these four dependent parameters no undisputed theoretical predictors are available. The theoretical predictors that have been proposed, either are based on questionable assumptions like external hypotheses or are applicable only for very small disturbances and hence their application to real rivers is doubtfull. Some empirical predictors are available, either developed especially for the Jamuna river or potentially applicable. From these methods a selection has to be made to identify the methods that are most suitable for use within the present Project for the prediction of the response of the river to FAP 22 measures.

7.4.2 Testing

Considering the above an important step in the selection procedure is the testing of the applicability of the various proposed methods against the characteristics of the Jamuna river. This holds in particular for the prediction of (1) the width of the river system, (2) the sinuosity, (3) the number of braids and (4) the total width of the river system. Some empirical predictors are available, either especially developed for the Jamuna river or potentially applicable. From these methods a selection has to be made to identify the methods that are most suitable for use within the present Project. Considering the above, an important step in the selection procedure is the verification of the applicability of the various proposed

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methods against data on the Jamuna river. The characteristics of the Jamuna river have been determined on the basis of satellite images. Some results of this analysis are presented in the Fig. 7.4-1 (notably the total width of the river and the width of the channels as a function of the chainage) and Fig. 7.4-2 (providing a relation between the number of channels and the total width of the river).

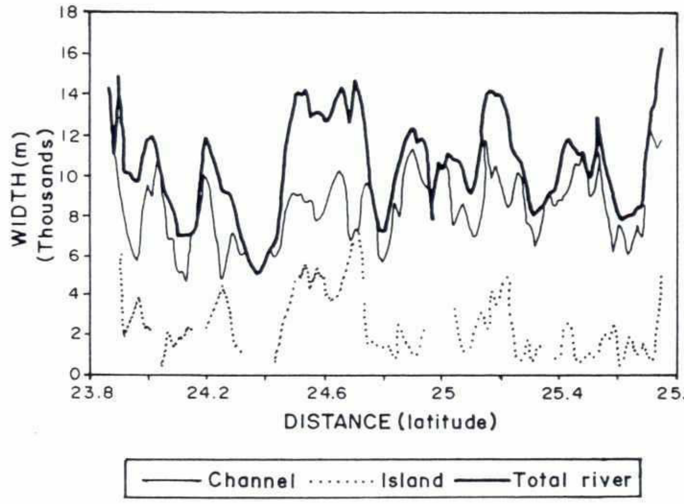

Fig. 7.4-1: Total width and combined width of all channels versus chainage of the Jamuna river for the year 1989

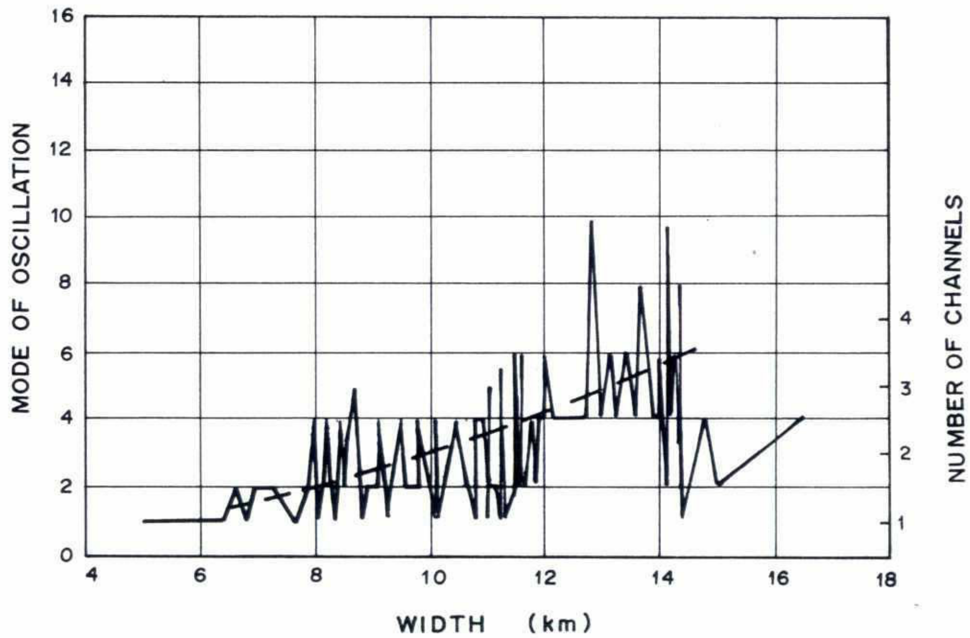

Fig. 7.4-2: Number of channels versus the total width of the Jamuna river

In a further step the different prediction methods have been compared with these river data. During this comparison both theoretical and empirical predictors were tested. The preference is of course for theoretical predictors, but in the experience of the Consultant even theoretical predictors often have to be calibrated using field data. If no theoretical predictors are applicable, it will be necessary to use empirical relationships.

If necessary, even empirical methods have to be developed especially for the Jamuna river. Hereafter prediction methods for (1) the width of the channels, (2) the sinuosity, (3) the number of braids and (4) the total width of the river system are considered.

Re (1) Width of Channels

The testing concentrated on the external hypotheses, according to Bettess and White (1987) the only methods (apart from empirical regime equations) to provide fair predictions for the channel width. The approach of Chang (1979) was being followed: the independent variables are (1) discharge, (2) sediment transport, (3) size of bed material and the number of channels is assumed to be 1. The method of Chang is based on the identification of a minimum slope. It was found here that no minimum is found when the influence of the walls is neglected and it is assumed that h = R (an acceptable assumption as far as the flow is concerned, because for B in the order of 1 km and h in the order of 7 m the hydraulic radius R virtually corresponds to the water depth H). Only via introducing the hydraulic radius a minimum is obtained, but as is shown in Fig.7.4-3 the location of this minimum depends very much on the schematization of the banks. In view of the very wide channels in the Jamuna river these results appear to be very doubtful. Fig. 7.4-3 is based on the combination of the Ackers & White (1973) sediment transport formula and the White et al. (1980) roughness predictor. Hence for large rivers, the Ackers & White sediment relationships together with the external hypothesis, predicts channel widths which are too small and channel depths that are too large. A similar result was obtained using the Engelund & Hansen sediment transport equation and roughness predictors. This does not necessarily imply that the external hypotheses is not acceptable in any case, but the results for Jamuna type of rivers indicate that the external hypothesis in combination with generally accepted sediment and hydraulic roughness predictors does not produce fair results.

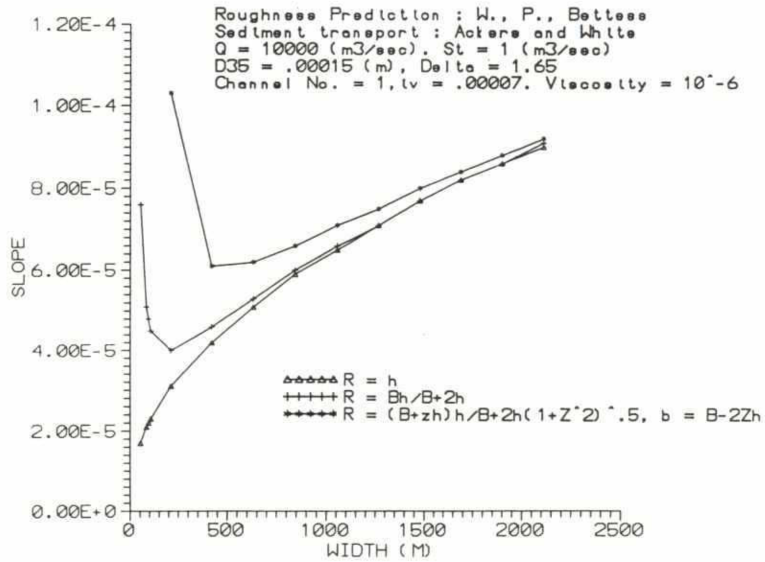

Fig. 7.4-3: Testing of external hypothesis using the Ackers & White sediment transport predictor and the White et al. roughness predictor

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Hence for the time being only an empirical relationship like a regime equation for the width can be used for the evaluation of changes in the river environment.

Re (2) Sinuosity

No independent theoretical prediction method for the sinuosity was identified other than the geometrical relationship given above as Equation (7.3-16). Hence for the time being this relationship will be used.

Re (3) Number of Channels

In Subsection 7.3.4 some methods for use within a prediction method were identified. All are basically empirical, apart from the method based on the marginal stability curve. Because of its potential as theoretical predictor this method was tested. Based on a given Q, the particle size D and the slope of the river i, the value of the predicted mode m was computed for different values of the width B. It was hoped that the value of m would correspond reasonably well to the observed values of m (see Fig. 7.4-2). The result for typical Jamuna conditions is presented in Fig. 7.4-4.

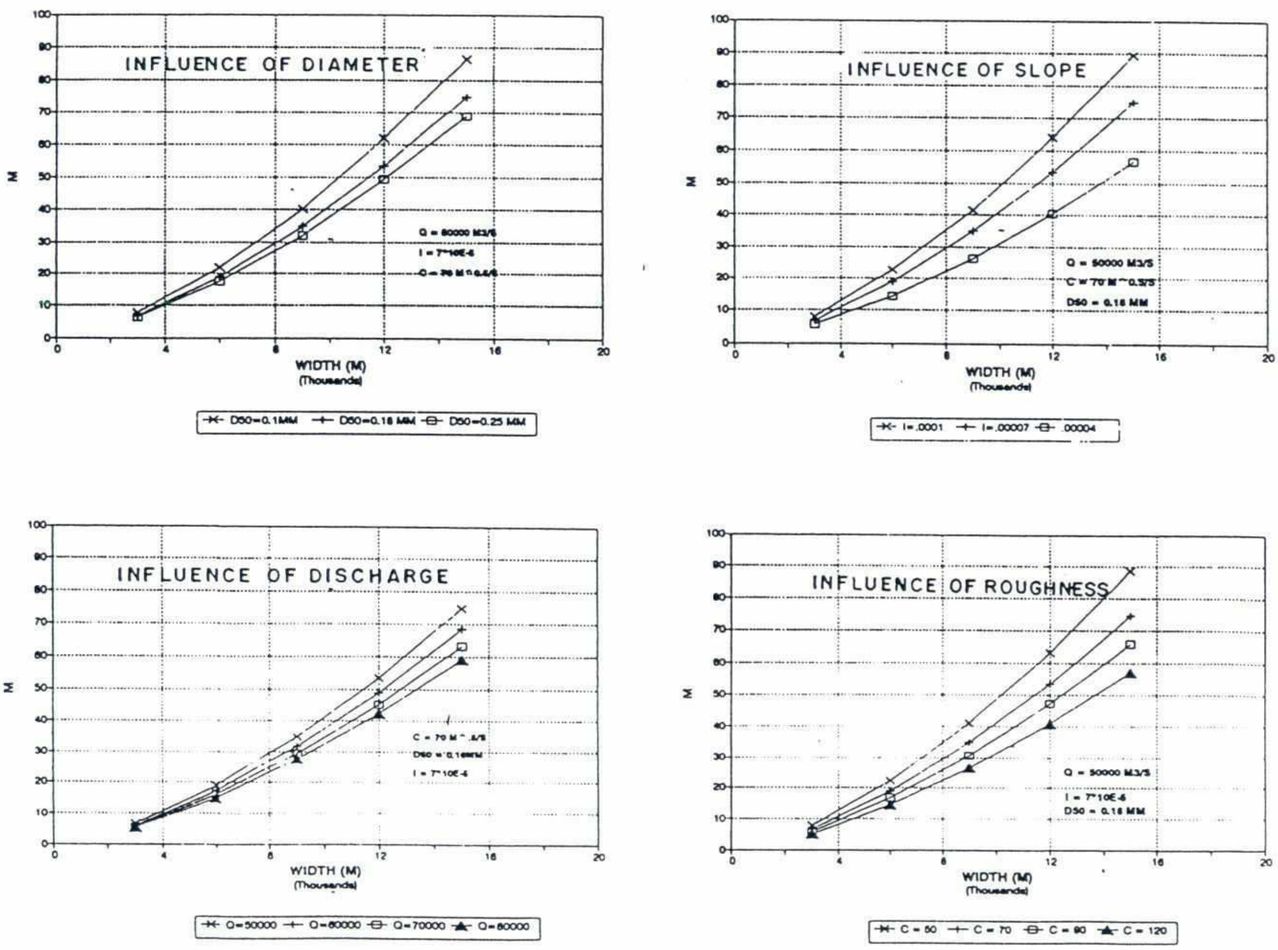

Fig. 7.4-4: Testing of prediction method for number of channels based on the marginal stability curve

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It can be concluded that the predicted mode m (being a measure for the number of channels) is much too high compared to the observed values (see Fig. 7.4-2). According to Subsection 7.3.4 a next step would be to relate the observed mode in the river (indicated in Equation 7.3-24 as m_*) to the predicted mode m, hence to develop a relation between m_* and m in accordance with Equation (7.3-24). This however, not considered as too fruitfull as then the theoretical predictor tested here would become just another empirical predictor and the advantage of using it would vanish. Hence this line of investigation was not further pursued.

The consequence is that no independent theoretical predictor for the number of channels could be found.

Re (4) Total width

No independent theoretical prediction method for the sinuosity was identified other than the geometrical relationship given above as Equation (7.3-24). That Equation, however, was developed for meandering rivers and it is difficult to visualize how it can be applied to braided rivers. The method of Ramette (1983) was also developed for meandering rivers, and cannot be used here in a straight forward way.

7.4.3 <u>Development of Prediction Method for Braided Rivers</u>

According to the Terms of Reference of the present study, a prediction method has to be selected applicable for the Jamuna river, but as can be concluded from the preceding subsection no method is available that straightforward can be applied to the Jamuna river. Hence it was imperative that a prediction method had to be developed especially for the Jamuna river conditions, as otherwise no prediction of the impact of certain strategies could be made. The philosophy behind the development of such a prediction method was that if and when possible a theoretically based predictor should be used, but if not available an empirical prediction would be applied. In the following the developed prediction method is described in some detail. The overview in the following table is meant (1) to clarify the steps taken in the development, and (2) to indicate which parameters were considered as independent and which as dependent, and which equations are used in the method.

The method applied here can be summarized as follows:

(1) Prediction of slope of river for given Q, V (the sediment transport integrated over the year) and D, via

(a) the continuity equation of the water, which in elementary form reads as:

$$Q = B\,h\,u \qquad (7.4\text{-}1)$$

but which for bankfull conditions and for k channels reads as:

$$Q_b = k\,U_b\,h_b\,B_b \qquad (7.4\text{-}2)$$

Step Number	Parameters			Equations			
	Independent	Dependent	Additional independent	Theoretical	Emperical	Additional theoretical	Additional emperical
1: fixed bed, width fixed, straight channel	Q, B, D, i_b	C, u, h		Continuity equation Equation of motion	Nikuradse roughness Chézy coefficient as function of roughness height		
2: movable bed, width fixed, straight channel	Q, B, D i_b	C, u, h, S		Do	Nikuradse roughness not applicable; instead to be used alluvial roughness predictor		
2a: movable bed, width fixed, straight channel but as dependent variable	Q, B, D, S	C, u, h, i_b		Do	Do		
3: as 2a, but with width as dependent variable	Q_b, D, V	B_b, C, u, h_b, u_b		Do	Do		Regime equation for width
4: as 3 but with curved channel	Q_b, D, V	B_b, C, u, h_b, i_b, p	i_v	Do	Do	Geometrical equation for sinuosity	
5: as 4, but with more than 1 channel	Q_b, D, V, i_v	B_b, C, u, h_b, i_b, p, k		Do	Do	Do	Number of channels k maximum
6: as 5	Q_b, D, V, i_v	B_b, C, u, h_b, i_b, p, k, B_t				Geometrical relation for total width	Meander length as function of e.g. discharge.

Table 7.4-1: Development of prediction method for response of Jamuna river

(b) the sediment transport equation via:

$$S_b = \frac{V}{\alpha_Q * 365 * 86{,}400} \tag{7.4-3}$$

$$s_b = \frac{S_b}{B_b\ k} \tag{7.4-4}$$

and

$$s_b = \alpha_s \sqrt{g\ \Delta\ D^3}\ \frac{0.05}{1 - \varepsilon} \left[\frac{h_b\ i_b}{\Delta\ D} \right]^{\frac{5}{2}} \frac{C^2}{g} \tag{7.4-5}$$

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(c) the equation of motion for the water which for uniform steady flow reads as:

$$U_b = C \sqrt{h_b \, i_b} \tag{7.4-6}$$

(d) a hydraulic roughness predictor:

$$C = C\ (h, i_b, \Delta\ , D, \ldots) \tag{7.4-7}$$

(e) a regime equation for the width, for which the Jamuna predictor is taken:

$$B_b = 16.1 \left(\frac{Q_b}{k} \right)^{0.53} \tag{7.4-8}$$

(2) Prediction of the sinuosity via:

$$p = \frac{i_v}{i_b} \tag{7.4-9}$$

(3) Prediction of the total width of the braided system via:

$$B_t = 2\, k\, A + B_b \tag{7.4-10}$$

where

$$\lambda = 10\, B_b \tag{7.4-11}$$

and λ (and α) are determined via the geometric relationships:

$$\alpha \text{ via } P = \frac{\alpha}{\sqrt{2\ (1 - \cos \alpha\ )}} \tag{7.4-12}$$

and

$$A = \frac{\lambda\ /\ 2}{\sqrt{2\ (1 - \cos \alpha\ )}} \ (1 - \cos \alpha\ ) \tag{7.4-13}$$

(4) Repeated computations for different values of k until the maximum value of k is found for which still the sinuosity p is (slightly) above 1.

The following remarks are made regarding the development of the method used here:

(a) Steps 1 through 2a correspond to the "normal", non-disputed approach, although it involves two empirical relations, notably an alluvial roughness predictor and a sediment transport predictor. The difference between step 2 and 2a is the selection of dependent and independent parameters.

(b) The steps 1 through 2 are valid for any value of the discharge. Accepting the slope of a river as a dependent variable can only be done if an equation for the slope is used. Here "dominant" conditions are considered and for the time being the bankfull discharge is accepted here as dominant discharge for the channel dimensions.

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Hence

$$B_b = f(Q_b) \tag{7.4-14}$$

(c) In the equations a factor α_Q is applied. This factor represents the influence of the hydrograph. Usually less sediment is transported over a whole year if the full hydrograph is considered compared with the case that the dominant discharge would be present during the whole year. Hence the sediment transport during bankfull discharge is determined by dividing the average sediment transport over the year by the factor α_Q, which is always less than 1.

(d) The calibration factor α_S takes care for the deviation of the actual transport compared with sediment transport predictors. For the Jamuna river is was found that α_S corresponds to about 2 (see ANNEX 2).

(e) The equation for the total width is derived from a geometrical relationship which is outlined in Fig. 7.4-5. It is felt that this corresponds to the minimum total width. The presence of permanent chars would increase the total width of the braided system. It would also mean a transition to a more anastomosing system.

(f) From the sinuosity the amplitude A of the curved channels can be determined. Also this is related to geometry as is in Fig. 7.4-6. The used equation holds only for real circles, but is probably a good approximation for low sinuosities.

(g) An important parameter is the wave length as this wave length determines the scale of the Fig. 7.4-5 and 7.4-6. It would be logic to relate the wave length λ to the bankfull discharge Q_b, but such a relation is not available. From the literature some relationships between the wave length λ and the channel width B_b are available (see e.g. Loopold et al, 1976 or Jansen, 1979). Here a relationship proposed by Leopold & Wolman (1960) is used. This relation reads.

$$\lambda = 10.1\ B_b^{1.01} \tag{7.4-15}$$

Here a simplified expression was used, reading

$$\lambda = 10\ B_b \tag{7.4-16}$$

being justified by the pre-feasibility stage of the present Project. It is stressed here that this formula in combination with the regime equation for the width in fact can be read as:

$$\lambda = f \left[\frac{Q_b}{k} \right]^{0.53} \tag{7.4-17}$$

Hence, for a given value of O_b the wave length λ is a function of k only.

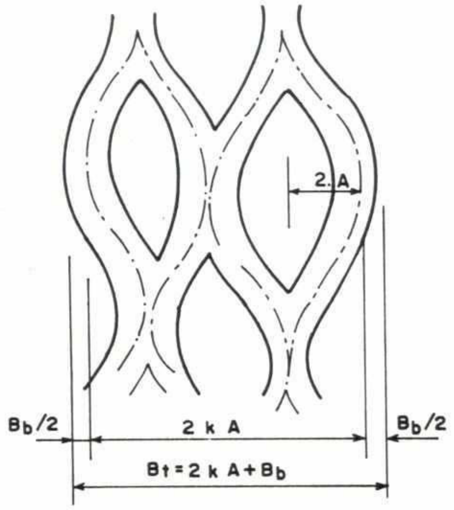

Fig. 7.4-5: Geometrical relation for the determination of the (minimum) total width

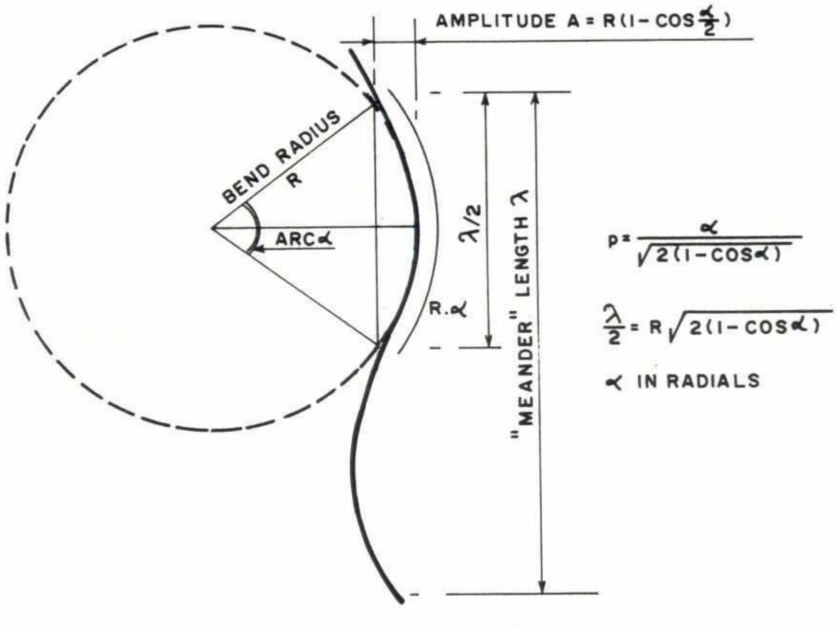

Fig. 7.4-6: Relation between amplitude and sinuosity for a curved channel

7.4.4 Calibration and Verification

The above method still needs to be calibrated before it can be applied to estimate the response of the river system to the river training and AFPM scenarios. Overviewing the method above the following independent input data are needed:

- bankfull discharge Q_b;

- frequency distribution of the discharges f(Q) to determine α_Q (for the time being it is assumed that all discharge is conveyed in the channels, neglecting the flow in the floodplain and over the chars; this is a good first approximation and would if assumed otherwise only result in a slightly different value of α_Q);
- yearly volume V or bed material load to be transported;
- characteristic particle size of the bed material D, and
- valley slope i_v.

The dependent variables for freely flowing river systems are the hydraulic roughness C (in principle a function of Q), bankfull depth h_b, the bankfull width B_b, the number of channels k, the average slope of the channels i, the sinuosity p of the channels, and the (minimum) total width B_t of the river. Checking the equations proposed above for the prediction of the river behaviour, it can be concluded that there is only one calibration parameter left, notably α_S. Hence a value of α_S has to be found that results in all dependent values to obtain fair values. This is less risky than it seems because for some of the dependent variables (roughness, sediment transport, width) relationships are proposed that have been derived on the basis of Jamuna data.

The calibration is carried out for the conditions at Bahadurabad. For this location the following independent data have been assumed:
- bankfull discharge Q_b = 44,000 m^3/s;
- frequency distribution of discharges conform ANNEX 4 of Interim Report 2 of FAP 1, yielding $\alpha_Q = 0.39$;
- yearly volume of sediment to be transported computed with the adjusted Engelund-Hansen formula (see ANNEX 2) yielding 140,000,000 m^3;
- characteristic particle size D_{50} = 0.215 mm;
- valley slope 8.5 cm/km.

In addition the Chézy coefficient was kept constant at 70 $m^{0.5}/s$, a value that is a fair approximation of the roughness during flood conditions (see ANNEX 2). Using the proposed method the dependent variables were computed. The results are shown in Fig. 7.4-7, where the main variable on the horizontal axis is the number of channels. The computations were done with a spreadsheet programme for different values of α_s. The calibration consists of selecting a value of α_S for which for the maximum value of k most of the river characteristics of the Jamuna river are conform the observations in the field.
These characteristics can be described as:
- number of channels of about 3 (see ANNEX 2),
- minimum total width about 10 km (see ANNEX 2),
- average bankfull depth according to the regime formula derived by Klaassen & Vermeer (1988a) which reads as:

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$$h_b = 0.23 \left[\frac{Q_b}{k} \right]^{0.32} \qquad (7.4\text{-}18)$$

is about 5 m for 3 parallel channels.

Inspection of Fig. 7.4-7 leads to the conclusion that a value of α_S of 2.7 corresponds to a maximum number of channels of 3. The corresponding total width is about 10 km, while the average depth of the channels is about 5 m. All these data are fairly in line with river characteristics as listed above, hence for the time being the value of 2.7 is accepted for the parameter α_S. An interesting aspect is that half the wave length of about 30 km, is obtained by using the present method. This value of λ corresponds to the length suggested by FAP 1 for the fixed points along the Jamuna river.

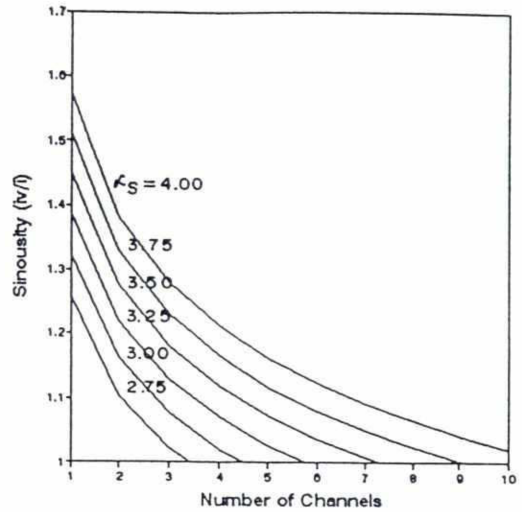
In a further step an attempt was made to verify the method proposed here. In first instance the conditions in the Jamuna river downstream of Sirajganj were simulated wit the proposed method, but in addition holding the value of λ_Q equal to 2.75. For the independent variables the following values were used:

- dominant discharge 44,000 m^3/s (same as for the conditions near Bahadurabad);
- frequency distribution of discharges same as near Bahadurabad, hence $\alpha_Q = 0.39$;
- yearly sediment volume 140,000 m^3 (also same);
- characteristic particle size 0.06 mm, and
- valley slope 5 cm/km.

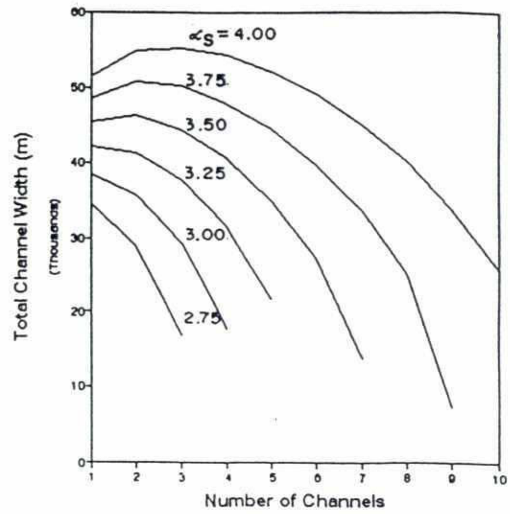
For these particular conditions it was predicted that the Jamuna river downstream of Sirajganj would have only 1 channel, with a meanderlength of some 90 km and a meander belt width of some 20 km. Comparing this with the actual conditions (2 channels on the average and a belt width of some 10 km, it has to be concluded that the tendency of decreasing number of channels is correctly predicted, but the actual prediction is less good.

To investigate this further the sensitivity of the prediction method for the input data was studied. It was found that the number of channels increases with the Chézy coefficient, the valley slope, the discharge and inversely related to the characteritic particle size (see Fig. 7.4-8). This is in line with the emperical predictors discussed in Section 7.3.4.

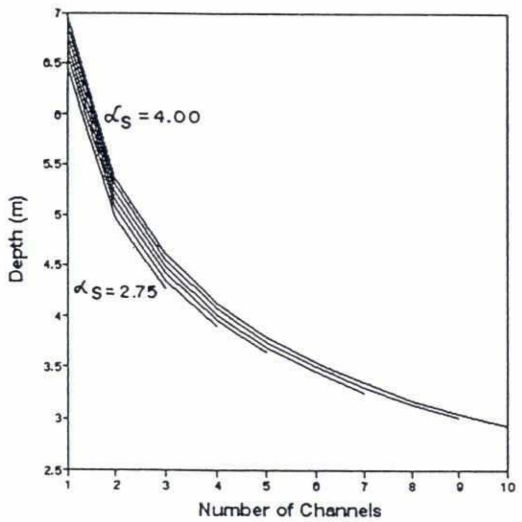
Finally an attempt was made to apply the prediction method to other rivers in Bangladesh. It was found, however, that the uncertainty as to the sediment load of these rivers, in combination with the possibility that different expressions for the different emperical equations should be used, did not allow to draw definitive conclusions as to the applicability of the other rivers.



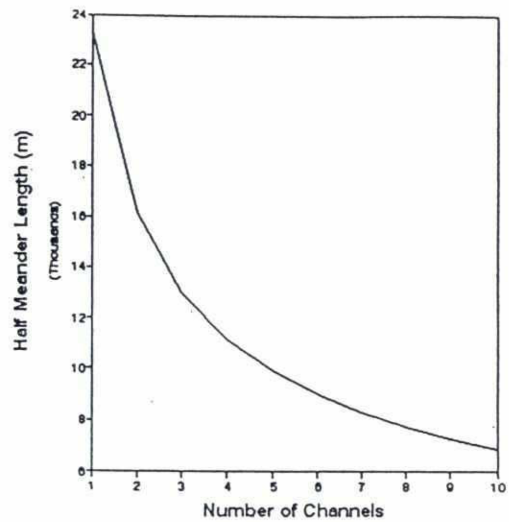
(a) Sinuosity



(b) Total channel width



(c) Average depth of individual channels



(d) Meander length

Fig. 7.4-7: Calibration of proposed method for Bahadurabad characteristics

Nevertheless considering that for Jamuna conditions a fair predictive behaviour was obtained, for the time being this predictive method has been applied to study the effect of scenarios for river training/AFPM.

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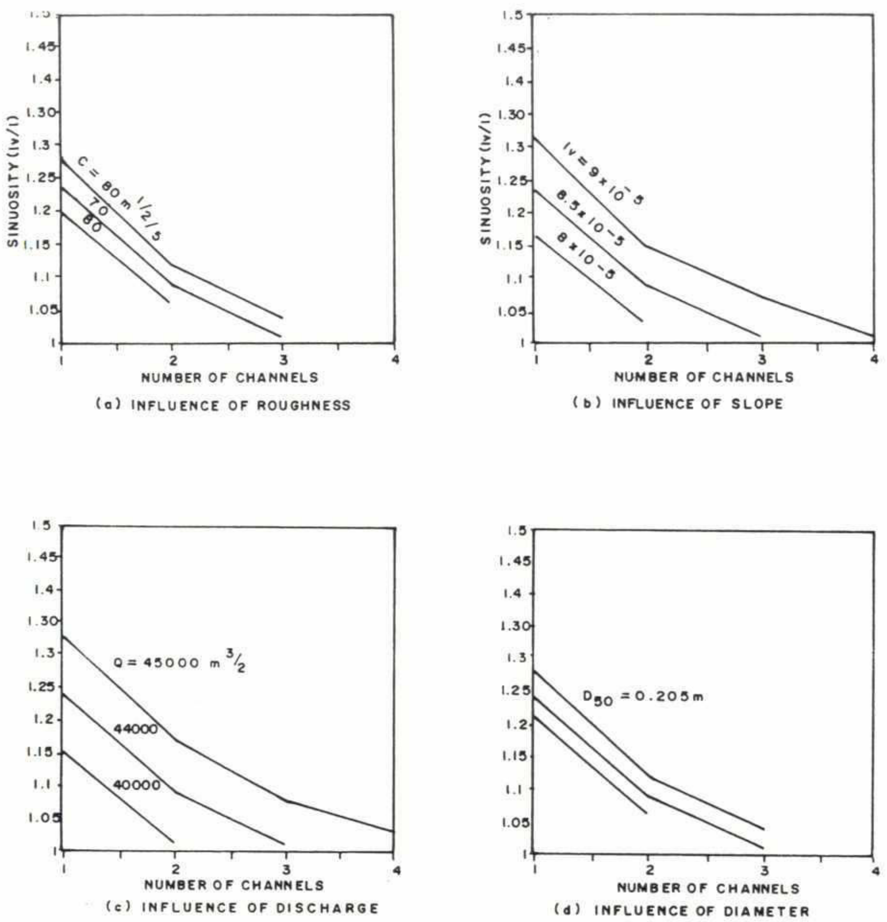

Fig. 7.4-8: Sensitivity of prediction for Bahadurabad for hydraulic roughness, valley slope, bankfull discharge and characteristic particle size

7.5 APPLICATION OF THE SELECTED METHOD

In this section the response of the different scenarios for river training/AFPM as identified in Chapter 5 of the Main Report is tentatively estimated using the method developed in the previous chapter. The three scenarios considered are described in Section 5.1 of the main report. The first two are aiming at an anabranched system, keeping the system of permanent chars intact. The third scenario aims at gradually reducing the size of the permanent chars by gradually reducing the width of the Jamuna river. In a further development the total width can be reduced even further. This will result in an increase of the attached char land.

The impact can be divided into:
(1) changes of the river banks, and
(2) changes in the channel and char system.
These are discussed in some detail hereafter, where the emphasis is on the ultimate hydraulic and morphological response.

In all three scenarios river training works (groynes and revetments) will be constructed, although the extent to which they are implemented varies considerably among the three scenarios. The consequence of the construction of river training structures is that the occurrence of steep eroding banks will be reduced and that the banks will increasingly become more "stony". "Stony" banks consists of gently sloping banks with boulders or concrete blocks as bank protection works. In between the river training works sedimentation areas may develop, characterized by gently sloping beaches. Scenario 1 is the most extreme in the substitution of the natural eroding banks by "artificial" environments, and the scenarios 2 and 3 are decreasingly less drastic.

Reducing the width of the channel and char system will have a more or less pronounced effect, depending on the width reduction. It is assumed that this reduction in width is hardly present in the scenarios 1 and 2. Only the third scenario ("Braided river with reduced width") may result in changes in the total width also. A distinction should be made between a reduction of the total width to a value of about 10 km and a value even lower that 10 km. Based on the analysis of the satellite images as presented in Section 2.3, it is clear that the total width of the river not including the permanent islands totals about 10 km. Hence a reduction of the width to this value will result in the disappearance of the permanent chars, but will not have a major effect on the channels as well. An estimate of this can be obtained from Fig. 7.4-7 in combination with Fig. 7.4-2.

The latter figure provides an emperical relation between the total width and the number of channels. Reducing the total width of the river to about 8 km results in a reduction of the number of channels to about 2. The characteristics of these can be read from Fig. 7.4-7 by reading on the vertical axes for the number of channels of 2. This yields:

- sinuosity of about 1.1;
- total width of about 15 km;
- meander length of about 33 km,

and a small increase in average depth. These are the characteristics of the natural channels if they would be allowed to develop.

The recurrent measures are however meant to keep the total width of the channel system to about 8 km, hence a reduction of the sinuosity to about 1.02 can be expected due to the recurrent measures. A reduction in sinuosity means that the geometrical fit between the upstream water levels and the level of the upstream bankfull levels (Equation 7-34) will no longer be present, hence the overall effect will be a small reduction in the slope of the river, corresponding to about 7% of the present slope. Hence the reduction in slope will be about 5 mm per km, resulting in a decrease in bed levels and hence in due time of the water levels in the Jamuna river of on the average a few decimeters. This will have both positive and negative effects; a reduction in water levels will reduce the flooding but it will also reduce the inflow into the Old Brahmaputra river. Because a narrowing of the river will be taken up, if ever, after several decades only, the change in slope will become noticeable even later.

Scenario	Impact on: River banks	Impact on: Channel and chars system
1. Anabranched system based on hard measures only	(1) Steep eroding banks will reduce in length substantially (2) "Stony" banks will replace these banks (3) Increased velocities near river training structures	No major impact
2. Anabranched system based on combination of hard and recurrent measures	Similar impacts as for scenario 1, but over a more limited reach length	No major impact
3. Braided river with reduced width	Similar impacts as for the scenarios 1 and 2, but only on a very small scale	(1) For a reduction to about 10 km: disappearance of permanent chars (2) For further reduction in addition: reduction of number of channels; increase of average depth and width of channels; "theoretical" increase in sinuosity compensated by a decrease due to the recurrent measures; minor changes in velocities

Table 7.5-1: Hydraulical and morphological impacts of the various scenarios

It is of interest to mention here that degradation of the bed is experienced in some river systems in Europe that have been subjected to narrowing in the past centuries. Other river systems, especially the Yellow river experience bed aggradation, mainly in the floodplain due to excess sedimentation. The ultimate reaction of the Jamuna river to river training/AFPM, as assessed here, may therefore be less than predicted here.

It is stressed again that the prediction made here is based on a method that has not yet been verified extensively. In a further stage of the project a more extensive verification is needed also versus characteristics of other rivers. Only when this verification yields acceptable results, the method can be used in further stages. It is stressed however, that the main results to be obtained later, will not yield completely different results.

It is hoped that the study component of FAP 24 will also tackle the prediction of the response of the river, based on larger insight on river systems of Bangladesh than presently available. Finally it should be realised that decisions on the preferred strategy for river training for the Jamuna river and the other main rivers have not to be taken now, but can await the knowledge to be assembled in the coming years.

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To facilitate easy access to the list of references below, the contents of the references have been categorized using the following codes.

G	=	general aspects of river engineering
t	=	contains a substantial section on river training works
v	=	supplies information on vanes
R	=	gives river engineering aspects of specific rivers outside Bangladesh
B	=	with special reference to Bangladesh rivers

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BANK PROTECTION AND RIVER TRAINING
(AFPM) PILOT PROJECT
FAP 21/22

DRAFT FINAL REPORT

ANNEX 2

RIVER MORPHOLOGY

DECEMBER 1992

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ANNEX 2

RIVER MORPHOLOGY

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1 INTRODUCTION

Bangladesh is the lowest riparian of the three great river systems of the region, the Ganges, the Brahmaputra and the Meghna having a large number of medium and minor tributaries.

The Brahmaputra river originates in a great glacier mass in the Kailas range of the Himalayans south of Lake Gunkyud in southwest Tibet (elevation 5300 m) and flows a total distance of 2880 km before emptying into the Bay of Bengal through a joint channel with the Ganges (Goswami 1985). The longitudinal profile of the Brahmaputra is presented in Fig. 1.1-1.

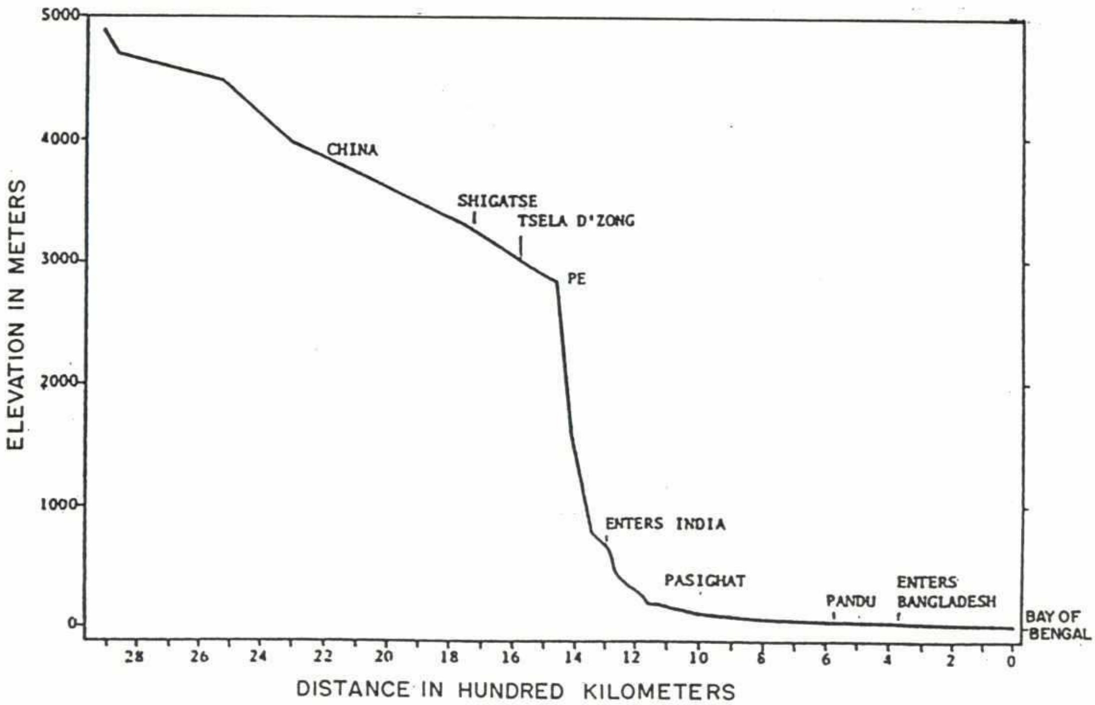

Fig. 1.1-1: Longitudinal bank profile of the Brahmaputra river

(Source: Goswami, 1985)

The length of the river in Bangladesh is 240 km. The Brahmaputra river after entering Bangladesh is named as the Jamuna or Brahmaputra/Jamuna and in Bangladesh it is a braided river.

About two centuries ago the Brahmaputra river shifted its original course to its present course as shown in Fig. 1.1-2. Since the avulsion the river has increased its total width and has generally shifted in western direction, in some reaches up to 10 km (Coleman, 1969 ; IECO, 1969).

The highly mobile river experiences large floods every year and its bed contains recently deposited loosely compacted fine sediment, the planform changes completely within a few years. The braided Brahmaputra/Jamuna river, having a large number of channels, is part of one of the largest river systems in the world.

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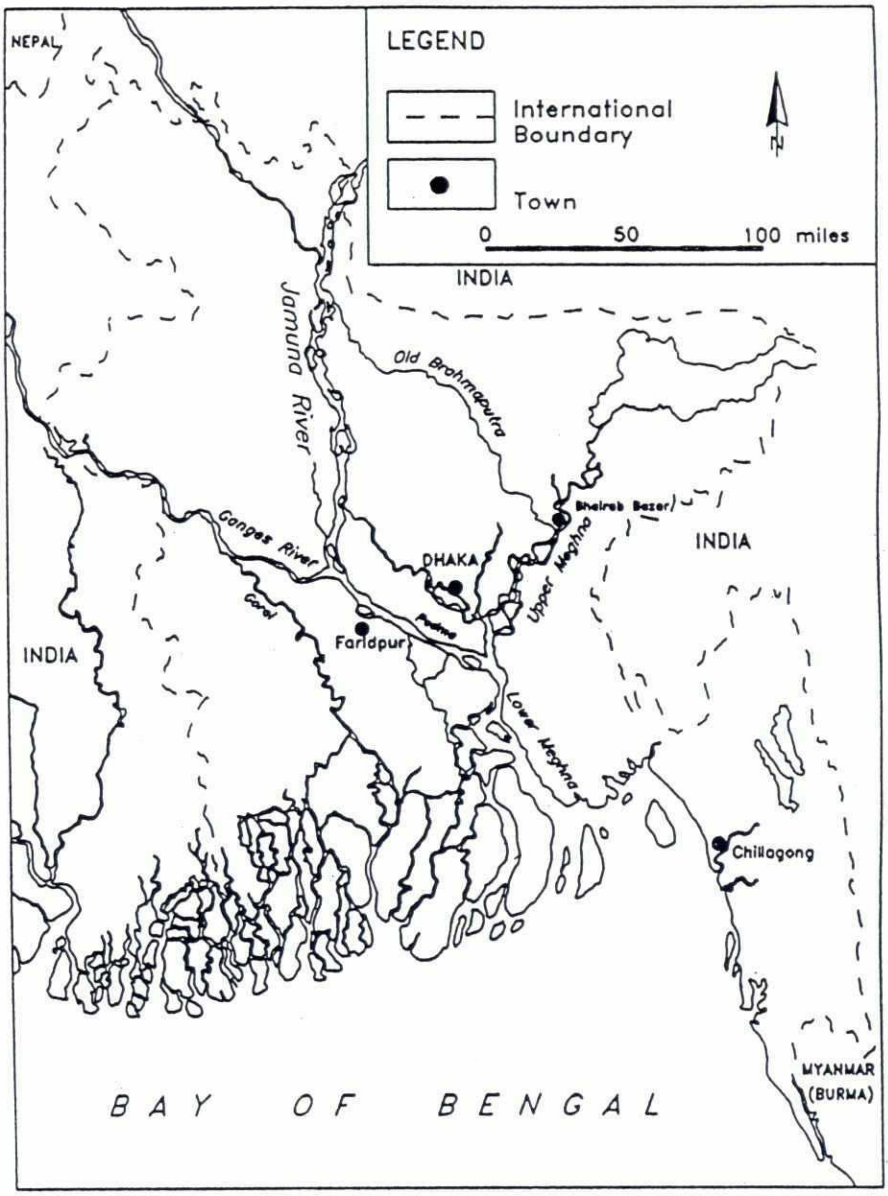

Fig. 1.1-2: Index map of Bangladesh showing major rivers

(Source : Klaassen and Masselink, 1992)

The river basin of the Brahmaputra/Jamuna includes part of China, Bhutan, India and Bangladesh. Of the total catchment area of 560,000 sq.km only 42,600 sq km is within Bangladesh territory. The valley slope in Bangladesh decreases gradually from 0.085 to 0.06 m/km. In general the width varies between 6 km to 18 km. The bed material is predominantly sand, quite uniform in size and average grain size varies from 0.25 mm in the upper reach to 0.16 mm in the lower reach. The bankfull discharge of the Jamuna river near Bahadurabad is about 44,000 m^3/s. Maximum peak discharge observed in 1988 was 98,000 m^3/s which is the highest ever recorded in the last 30 years (China-Bangladesh Joint Expert Team, 1991). Average annual flow of the Jamuna is 19,000 m^3/s and during low flow discharge is between 4,000 to 12,000 m^3/s. Average annual runoff of the Brahmaputra/ Jamuna river is about 618 X 10^9 m^3/s, the bulk of this occurs in the monsoon season (June to September). The mean annual sediment load amounts to about 140 million m^3.

The discharge hydrograph of the Brahmaputra/Jamuna river commonly displays two or three major flood peaks as in Fig. 1.1-2. Colemen (1969) described those as follows:

"The first flood peak, if present, generally occurs during Mid-June and is characterized by an extremely rapid increase in discharge. Within a few days the discharge may increase from less than 17,000 m^3/s to over 45,000 m^3/s. After this rapid initial rise, the river level generally falls slightly, and then in the latter part of July or the first part of August the major flood peak occurs. This flood period will generally last for nearly a month, then the river will again drop slightly. During the first part of September, a third flood peak will occur. In the latter part of September or first of October the river stage will begin to subside. The water-level hydrograph will also commonly show these peaks, especially during a low-flood year. The water level hydrographs also illustrate the rapid rise and fall of water levels associated with the flood stage".

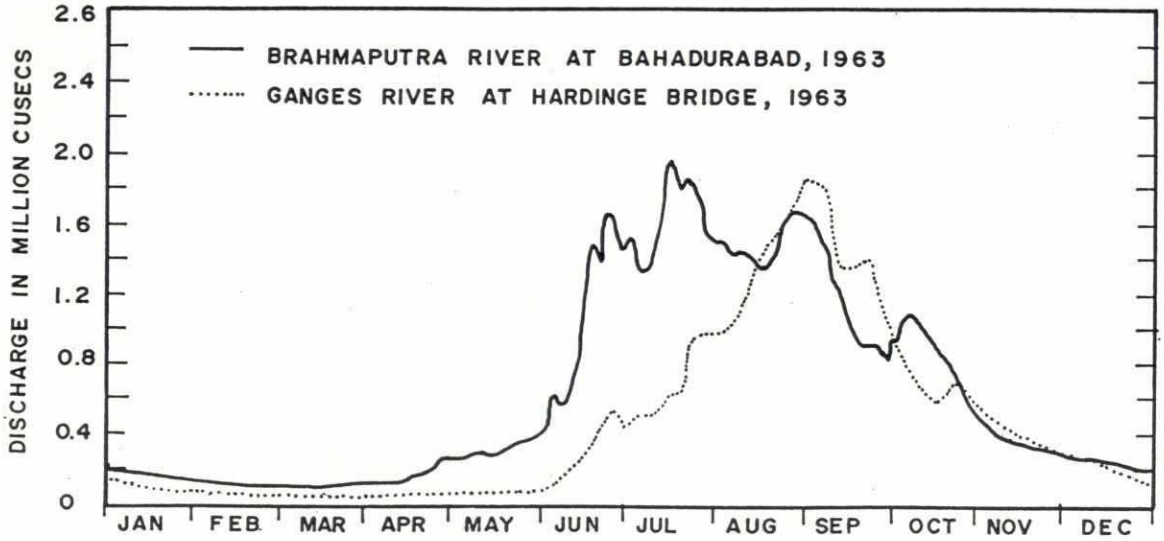

Fig. 1.1-3: Discharge hydrographs of Brahmaputra and Ganges rivers

(Source: Plotted after data in International Engineering Company, 1964)

The morphological characteristics of the river have hardly been studied in the past, therefore, our theoretical understanding of the river is still fairly limited. Most of the informations available from different project reports are in the form of mostly empirical relationships. The first comprehensive description of the Brahmaputra/Jamuna was given by Coleman (1969). Since then much more data of the river have been collected partly by regular field measurments of BWDB, several other studies, Bristow (1985), Klaassen and Vermeer (1988 a, b), Klaassen and Masselink (1992) and more recently via satellite images which are more and more proving to be an indispensable tool for deepening insight into the complex morphological process of the braided sand bar river system. The present study utilized additionally collected information from the following projects in an integrated manner:

- Jamuna Bridge Study, Phase 1 and Phase 2
- Brahmaputra River Training Studies (FAP 1)
- China-Bangladesh Joint Expert Team, 1991 (CBJET)
- Char Study (FAP 3.1).

2 GEOLOGICAL SETTING AND HISTORICAL CHANGES

2.1 GEOLOGICAL SETTING

Bangladesh consists primarily of Quaternary alluvial deposits laid down by the Ganges, the Brahmaputra and the Meghna Rivers. Geologically speaking the Bengal basin is bordered on the west and northwest by lower Jurassic trap rock in hills averaging 150-250 m above sea level. On the northeast lies Shillong Hills, which is predominantly composed of Eocene sand stones and Numilitic limestones. Elevations of the plateau approach 1400-1800 m above sea level. The eastern boundary is formed by the Tripura and Chittagong Hills. The southern boundary is formed by the Bay of Bengal.

According to Coleman (1969) the largest part of the basin is formed by Quaternary sediments deposited by the Ganges, the Brahmaputra and the Meghna rivers and their numerous tributaries and distributories. In this region relief is very low measuring a few tens of a feet. The pleistocene alluvial terraces are restricted to four major regions. Coleman has shown their distribution in the following Fig. 2.1-1.

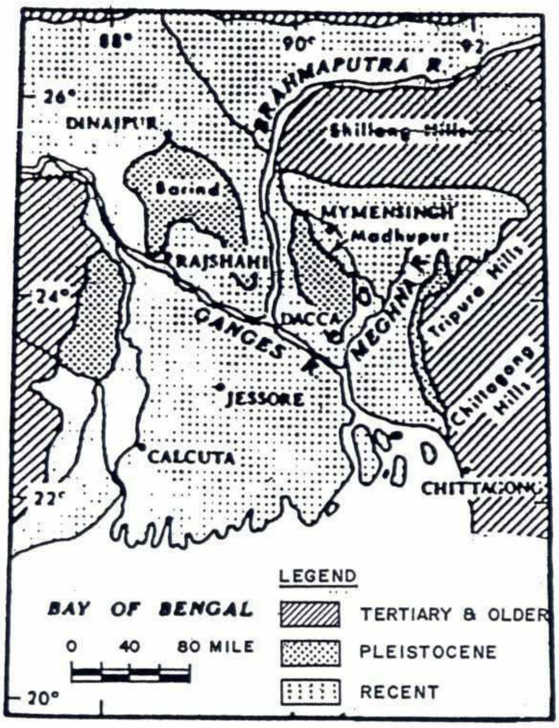

Fig. 2.1-1: Generalised geological map of Bangladesh

(Source: Coleman, 1969)

Of the four areas, two flank the basin on the east and west and the other two lie within the basin. These latter two areas are known as the Barind and Madhupur jungle. The pleistocene sediments were deposited as floodplains of the former Ganges-Brahmaputra river systems. The sediments are highly oxidized and are typically reddish-brown or tan. Because of their antiquity and long periods of exposure, they are characteristically more compacted and weathered than are more recent floodplain sediments. In many areas, the boundary separating the recent and pleistocene sediments is sharp and quite straight. These boundaries are the result of recent tectonic activity.

The remaining region of Bengal basin consists predominantly of recent alluvial and deltaic sediments. In the upper valley (land above tidal inundation) these deposits are composed primarily of silts and sands; in the lower delta they are principally silts, clays, and peats. Because they were recently deposited, they contain a high water content, are commonly loosely compacted, and are generally gray. The shifting nature of the Ganges and the Brahmaputra rivers make the geographic relationships of various sedimentary units highly complex.

The China-Bangladesh Joint Expert Team (1991) has divided Bangladesh in five geomorphological zones. These are shown in the following Fig. 2.1-2.

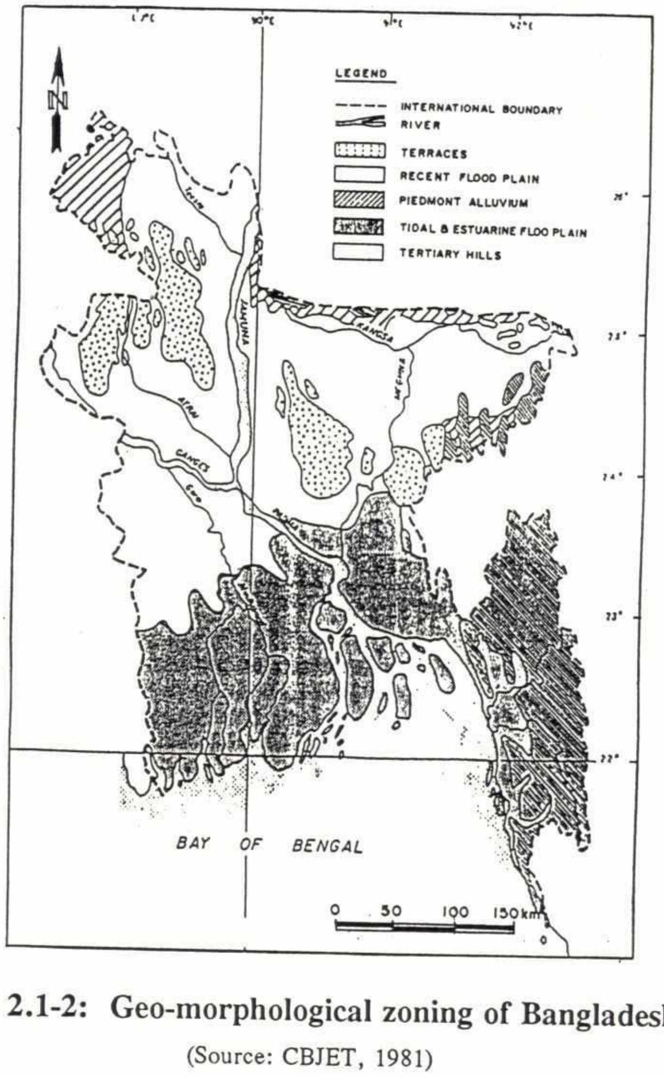

Fig. 2.1-2: Geo-morphological zoning of Bangladesh

(Source: CBJET, 1981)

JICA conducted geological surveys and field borings along the Brahmaputra during 1973-76. On the basis of these data CBJET team drew a geological profile which is presented in Fig. 2.1-3.

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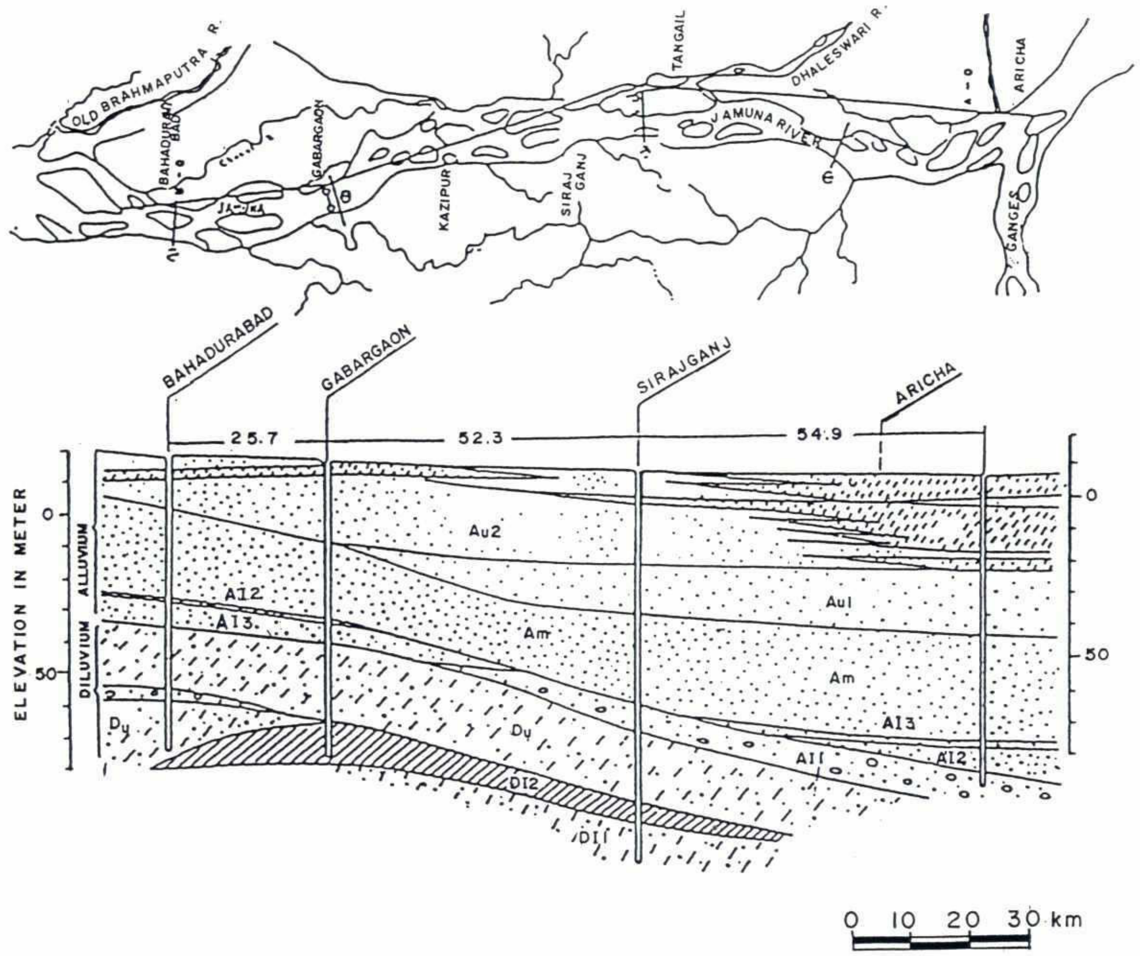

Legend

A u 2 - Silty loam, clay fine sand
A u 1 - Mainly mid-sand
A m - Medium sand
A13 - Sand Gravel layer
A12 - Gravel
A11 - Bottom gravel
Du - Gravelly soil
D12 - Clay or Mud stone
D11 - Gravelly soil

Fig. 2.1-3: Geological profile along the Brahmaputra/Jamuna river

(Source: CBJET, 1991)

According to the CBJET (1991) the Bengal basin is a tectonic basin that has undergone the co-action of two tectonic forces. The northward thirst caused by the northward movement of Indian Plate and the vertical pressure by the weight of the deposits. The vertical pressure promotes subsidence as well as further sedimentation. According to them Bangladesh can be tectonically divided into three regions. These are:

(a) Stable shelf
(b) Bangladesh fore deep
(c) Folded belt

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The tectonic classification of Bengal basin is shown in the following Fig. 2.1-4.

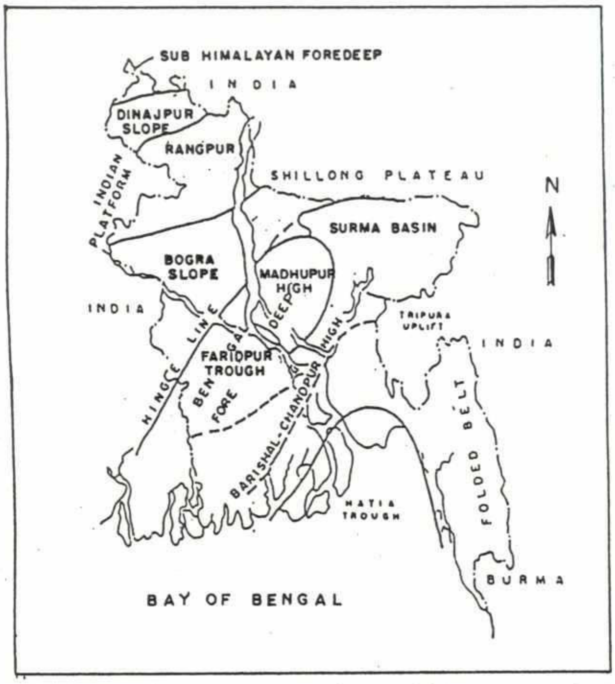


Fig. 2.1-4: Tectonic classification of Bengal basin

(Source: CBJET, 1991)

In Bangladesh the alluvial plain deposits are sorted in horizontal distribution. The grain size gradually becomes smaller and the thickness of deposite gets larger downstream and south eastwards. This can be seen from the Fig. 2.1-5.

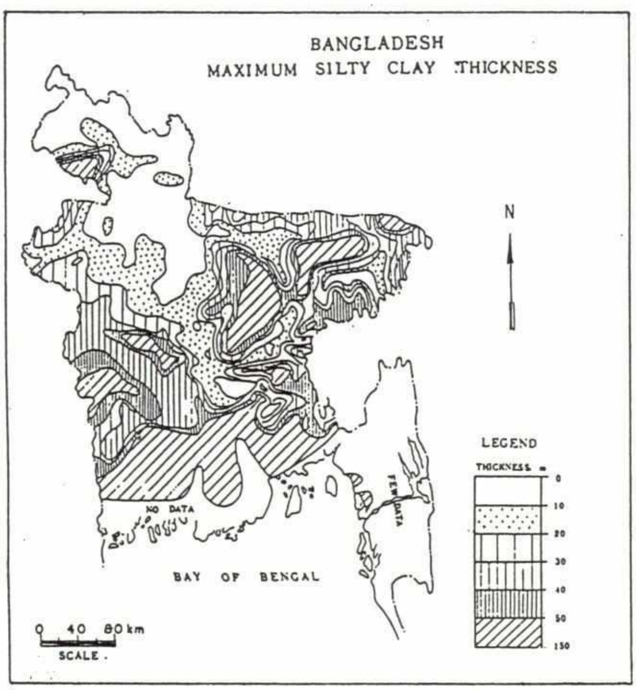


Fig. 2.1-5: Thickness of deposite

(Source: CBJET, 1991)

2.2 HISTORICAL CHANGES

The history of great changes of the river courses of the Brahmaputra/Jamuna are more than 2000 years as can be seen in Fig. 2.2-1.

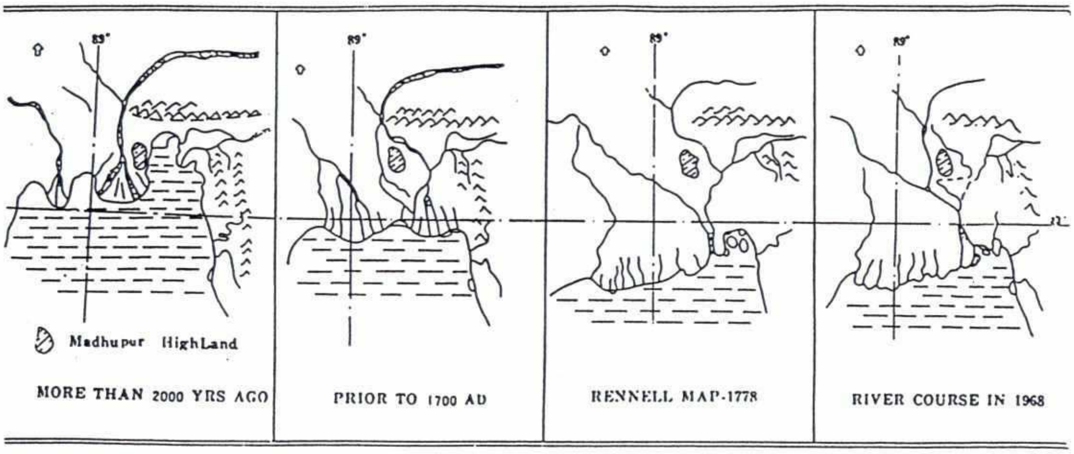


Fig. 2.2-1: Land formation and river courses in Bangladesh

(Source : CBJET, 1991)

The land formation and river course maps show that the Brahmaputra/Jamuna river had followed four changing courses in the last 2000 years.

Rennell's map published in 1778 reflects the first reliable information on the planform of the Brahmaputra/Jamuna river. It shows a braided river taking a south easterly alignment after flowing over the Shillong hills in India and thereafter follows the eastern edge of the Madhupur tract. The geomorphological map published by Coleman (1969) shows that the river had previously followed a course further to the north-east, 80-100 km east of its present course in 1765. A map published in 1830 by Wilcox is considered to provide a reliable picture of the planform of the river, a few year after its avulsion between 1780 to 1830 (FAP 1, Main Report).

Simplified and rough interpretation of 'Land Formation and River Courses' (Fig. 2.2-1) reflects that the Brahmaputra river channel has changed back to west of Madhupur high land and flowed along approximately the present course since 1830 (CBJET, 1991).

Quite different interpretations persist for the reason or period of avulsion of the Brahmaputra/Jamuna river from east to west of Madhupur highland prior to 1830. Though many historic documents relate to the time of this change in river course, but little agreement exists among them as to exact date. La Touche (1919) suggested that the change was rapid, occurring in 1787 as a result of a rapid increase in water volume in the Brahmaputra. Hirst (1916) indicated that the change took place gradually between the years 1720 and 1830. Mahalanobis (1927) agreed with La Touche and claimed that a catastrophic flood occoured in 1787 which caused the Brahmaputra river to abandon its old course. In all probability the diversion of the Brahmaputra was gradual, as most major river diversions are.

From the informations mentioned above it is reasonable to believe that in historical times the river has flowed along the present old Brahmaputra course but between 1780 to 1830 it has shifted to its present course. The CBJET (1991) study concludes that the Brahmaputra/ Jamuna river would be stabilized at its present location in a long term if large natural changes does not take place, probability of changes in river course in large scale would be very little. This study, as an answer to explain the present widening nature of the river, suggests that the Brahmaputra/Jamuna river systems is located in one of the tectonically most active regions of the world, hence the possibility exists that the river is not yet fully adjusted to this new condition.

2.3 PRESENT DAY TREND

To understand short term changes of river banks of the Brahmaputra/Jamuna river since 1830, Coleman (1969) compared thirteen available maps and selected maps between 1944-1963 as reliable ones. Within the period bankline migration was found not predictable but rather it was erratic. However overall indication was that a higher number of locations on the right bank showed erosion than on the left bank. Further study to detect a preferred direction of migration revealed that the river has been migrating in a westerly direction since 1830.

FAP 1 study used maps published by Rennel in 1765, Wilcox in 1830, Survey of India 1914, Survey of Bangladesh maps issued in 1951-57, 1967-69 and 1978-79 and all available Landsat images to detect migration of bank line. Analyzing general pattern of bank erosion, the study put the following comments :

"On a geological time scale the Bahmaputra/Jamuna river is drifting westwards and since this is most probably associated with tectonic trends, it may be anticipated that the drift will continue until it encounters the less erodible material of Barind tract at the maximum probable average rate of 50 m/year".

For the comparison of the bankline of the Brahmaputra/Jamuna in 1989 with that of 1830, CBJET stated that the bankline to the right of the reach retreat in large scale is due to erosion in 159 years. During a same period, on the left bank only limited amount of retreat of bankline was found and for the remaining sections there was deposition as can be seen in Fig. 2.3-1.

CBJET (1991) in order to analyse the magnitude of migration of banklines, divided area of deposition or erosion by 206.33 km, the lenght of the reach of the Brahmaputra/Jamuna. The results are given in Table 2.3-1.

The Table 2.3-1 also indicates that average annual rate of retreat of the right bank line between 1834-1952 was 44.5 m and between 1952-1989 was 67.3 m. So the study concludes that there is a trend of increase in the rate of retreat of the right bank in recent years.

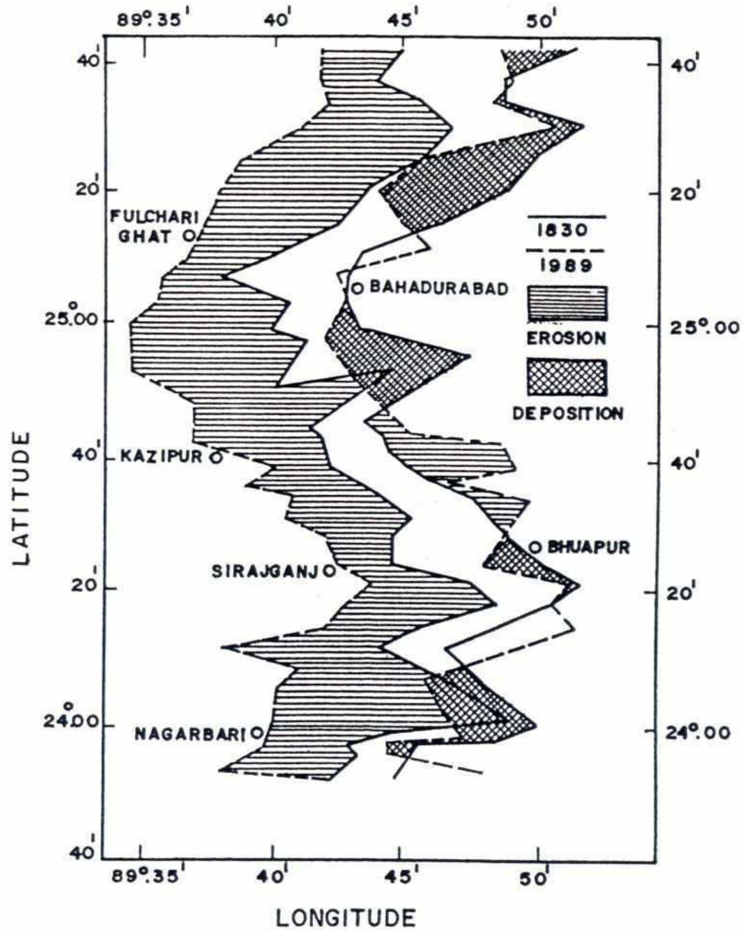


Fig. 2.3-1: Comparision of bankline in 1830 & 1989 of Brahmaputra river

(Source: CBJET, 1991)

+ deposition - erosion

Period	Right Bank			Left Bank		
	Erosion/ deposition area (km²)	Average erosion/ deposition width (m/yr)	Annual erosion/ deposition width (m/yr)	Erosion/ deposition area (km²)	Average erosion/ deposition width (km)	Annual erosion/ deposition width (m/yr)
1830-1952	-1,121.37	-5.43	-44.5	+147.45	+0.71	+5.82
1958-1989	-513.63	-2.49	-67.3	+18.43	+0.09	+2.43
1830-1989	-1,635.0	-7.92	-49.8	+165.88	+0.80	+5.03

Table 2.3-1: Average erosion/deposition length of Jamuna river bank

(Source : CBJET, 1991)

From a comparison of the central line of river channel in 1830 with that of 1989, the CBJET study found that the whole river has migrated 4.2 km on the average to the right. Shifting rate of channel central line in recent years is greater than in early period, an observation quite consistent with the variation of retreat rate of banklines.

A detailed analysis was carried out by FAP 19 to measure average annual movement of the main bank line of the Brahmaputra/Jamuna river for FAP 1 study. FAP 3.1 study made use of these time scale satellite imageries based on twenty years Landsat data and prepared Fig. 2.3-2 to represent the Jamuna left bank and right bank annual movement respectively.

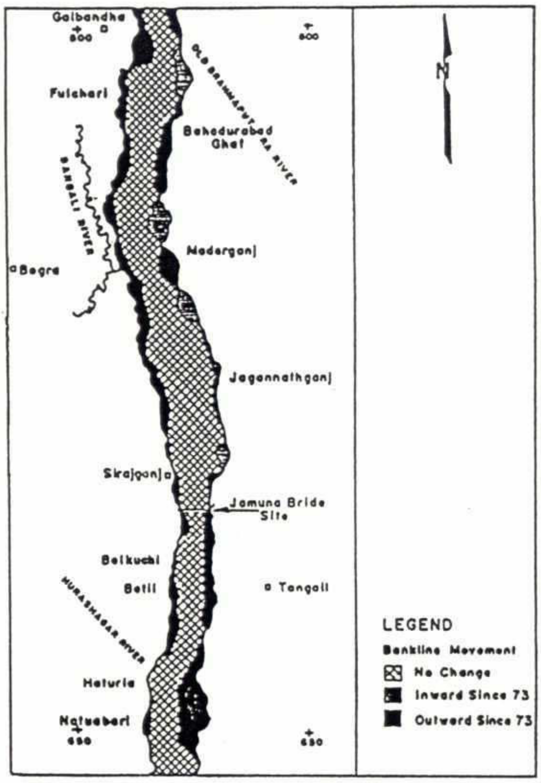


Fig. 2.3-2: Main bank erosion and accretion 1973 to 1992
(Source : SOGREAH, et al., 1992)

From the figure it is evident that the right bank has eroded westward along its total length at an average rate up to 150 m a year over the last twenty years. The situation on the left bank is different having been subjected to alternate erosion and accretion up to 300 m average in some places and 250 m in others.

2.4 INFLUENCE OF TECTONICS

The Bengal basin, a Ganges-Brahmaputra river delta is situated at the south of the Himalayans and is counted to be one of the most active tectonic zones in the world. Studying the earthquake characteristics recorded in and in the surroundings of Bangladesh since 1900, Coleman (1969) identified a definite alignment of earthquake epicenters along the east valley wall of the Bengal basin and in one case the epicenter was recorded within the vicinity of the Brahmaputra river. All these earthquakes were found to point to a definite zone of weakness. The National Geographic Society published a map in 1967 which also tends to show this zone of weakness. A report published in the 'Civil Engineer' (H. Brammer, personnel communication 1967) indicated that a large section of railroad and embankment sank in underlying soil during the earthquake in 1897 in Bangladesh. Earthquake vibration generally results in liquefaction of loosely compacted silt and sand layers and during the process extrusion of sand to the surface along small fractures occur. Coleman (1969) reported that local inhabitants claim that Madhupur highland was formed during earthquakes in 1897 and 1935.

Coleman (1969) suggested that probably faulting was the major cause of the most recent shift of the Brahmaputra/Jamuna river from its course east of Madhupur to its present position. CBJET (1991) opined the same view stating that other than the impact of an earthquake, the change was due to the squeezing of fault blocks which resulted in a rising of the old Brahmaputra river bed.

All these informations indicate that tectonic activity has played an important role in controlling the morphology of the Brahmaputra/Jamuna river.

3 SEDIMENTOLOGY

3.1 BED MATERIAL

Bed Material characterstics of the Brahmaputra/Jamuna river were studied by Coleman (1969). He presented data on grain size analysis of bed materials of seventytwo samples of the Brahmaputra/Jamuna river collected from throughout its length in Bangladesh and indicated an average grain size value of 0.172 mm.

The Jamuna Bridge Appraisal study also reviewed the results of bed material of the Jamuna river. The results were collected from RRI (BWDB) and analyzed by station and by year. Fig. 3.1-1 represents the average grain size (D_{50}) of bed material together with standard

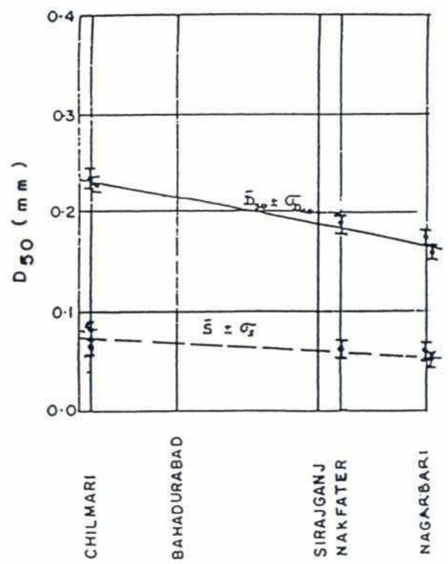


Fig. 3.1-1: Average grain size and gradation of bed material of the Jamuna river

(Source : RPT, NEDECO, BCL, 1987)

deviation and approximate gradation. The bed materials were found to be uniformly graded fine to medium sand and D_{50} value ranges between 0.235 to 0.165 mm from the upper to the lower reach.

According to FAP 1 study, bed materials collected from Sirajganj and Kazipur (nearly the lower reach) in the Brahmaputra/Jamuna river, the average size ranges between 0.15 to 0.17 mm.

An extensive study was carried out by CBJET in 1991 on bed materials of the flood plain and the main channel of the Brahmaputra/Jamuna river. The average grain size (D_{50}) of the materials are listed below (Table 3.1-1).

Item	Location	Brahmaputra/Jamuna river		
		Upper Reach	Middle Reach	Lower Reach
Bed Material	Flood Plain	0.19	0.18-0.19	0.17
	Main Channel	0.26	0.20-0.25	0.17

Table 3.1-1: Average grain size of bed material of the Brahmaputra/Jamuna river
(Source : CBJET, 1991)

It appears from the table that the D_{50} value for the main channel has gradually decreased from 0.26mm in the upper reach to 0.17 mm in the lower reach, which fairly relates to observations made by all other studies.

3.2 HYDRAULIC ROUGHNESS

The hydraulic roughness of the Jamuna river was determined by two methods under the Jamuna Bridge Project Phase 1 study using historical data collected by BWDB, as presented below:

- the first method 'via discharge measurement' makes use of local average water depth, flow velocity in combination with measured slope over a longer distance, and
- the second method, 'via average at a station' makes use of relationships between average depth versus discharge and channel width versus discharge.

For the first method, the hydraulic roughness as presented by the Chezy coefficient:

$$C = Q/B\ (h)^{3/2}\ i^{1/2}, \qquad (3.2\text{-}1)$$

in which Q = discharge (m^3/s), B = stream width (m), h = average depth (m) and i = slope (-) is computed using data presented by Uddin (1985). The results for station Sirajganj are given in Fig. 3.2-1. From these data it is concluded that Chezy values vary between 40 $m^{1/2}/s$ for low flow to 100$m^{1/2}/s$ for flood conditions. The very high Chezy values for flood conditions indicate a transition to flat bed for higher discharges. However, based on the study of bed form dimensions 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. There is of course the possibility that the large Chezy values are the result of a combination of local and general river phenomenon.

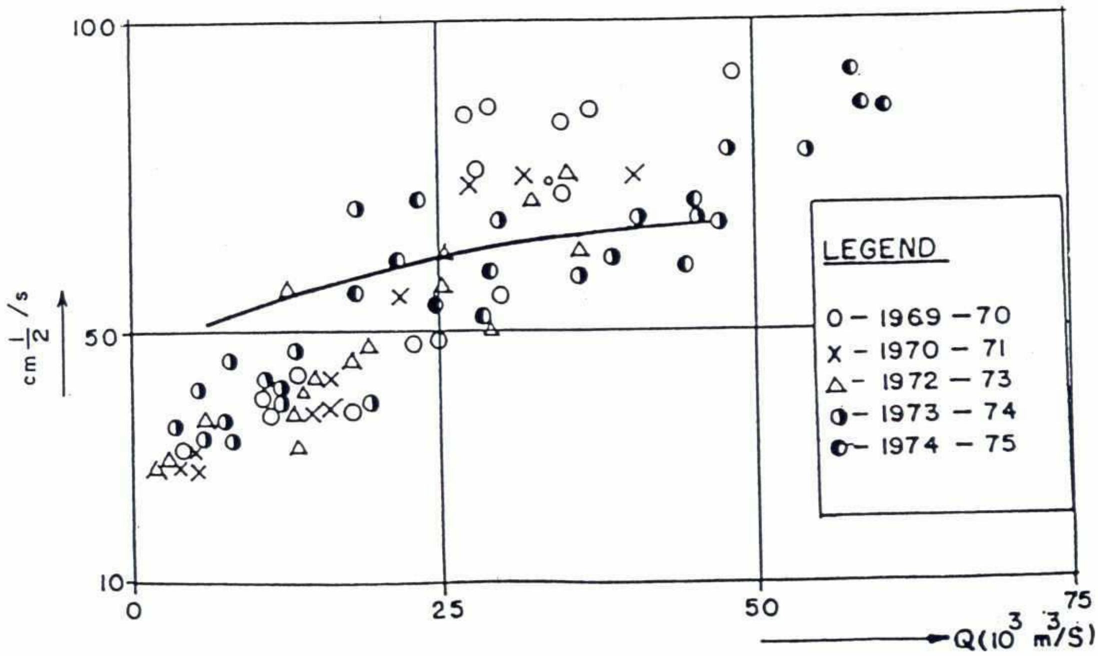


Fig. 3.2-1: Hydraulic roughness for the Jamuna at Sirajganj
(Source: RPT, NEDECO, BCL, 1989)

The second method is based on the analysis of cross-sectional data. This analysis aimed primarily at the derivation of equations given by B/Q and h/Q relations for the Jamuna river (see Klaassen and Vermeer, 1988). With these average at-a-station equations an estimation of the Chezy coefficient can be obtained in the following way:

1) the general expression for the equations reads:

$$B = a\, Q^b, \tag{3.2-2}$$

$$h = c\, Q^d, \tag{3.2-3}$$

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

2) insert equations (3.2-2) and (3.2-3) in equation (3.2-1), yielding:

$$C = \frac{Q^{1-b-1.5d}}{a.c^{3/2}i^{1/2}} \qquad (3.2\text{-}4)$$

For the Jamuna river the following approximate values for the parameters were found, a = 18.9, b=0.51, c=0.56, d = 0.23 yielding the following expression for the Chezy coefficient:

$$C = \frac{0.126\ Q^{0.14}}{i^{1/2}} \qquad (3.2\text{-}5)$$

An assumed slope of $i = 7x10^{-5}$ yields a Chezy coefficient (see also Fig. 3.2-1) varying between 47 $m^{½}/s$ at a discharge of 4000 m^3/s and 67 $m^{½}/s$ at a discharge of 44,000 m^3/s, which is approximately bankfull discharge.

The results of the second analysis seem to yield a more realistic Chezy coefficient. In any case, also this approach points at a substantial increase in Chezy coefficient for higher discharges.

3.3 SEDIMENT TRANSPORT AND SEDIMENT EXCHANGE

It is quite difficult to have an accurate prediction of sediment distribution and yield in a braided river like the Jamuna for all the periods, especially during Monsoon due to the complex variation of sediment load with incoming flow.

Jamuna Bridge Appraisal (JBA) study made a comparison between measured and predicted sediment load in the Jamuna river at Bahadurabad station (Fig. 3.3-1).

Three total bed material load predictions were used for the comparison. It followed that measured coarse suspended load is in excess of the predicted rates by a factor 1.5 from Van Rijn (1987), 3 for Ackers and White (1973) and more than 5 for Engelund/Hansen (1967). Jamuna Bridge Project Phase 2 study suggests that Engelund/Hansen transport formulate multiplied by 2 gives a fair prediction of the bed material transport of the Jamuna river where use was made of a regime equation.

Dune tracking measurement was conducted in the Jamuna to determine bed load transport during the flood of 1987 under the JBA study. From the data obtained it was observed that bed load (transport) is 10% of the suspended bed material transport, indicating that the most important mode of transport is suspension. This finding fairly corresponds to a prediction by Van Rijn (1987) and the CBJET (1991) study also confirms the relationship.

The CBJET study made a correlation analysis for the transport rate of measured sediment load and measured discharge from 1968 and 1980 for the Jamuna and established an average correlation coefficient of 0.887.

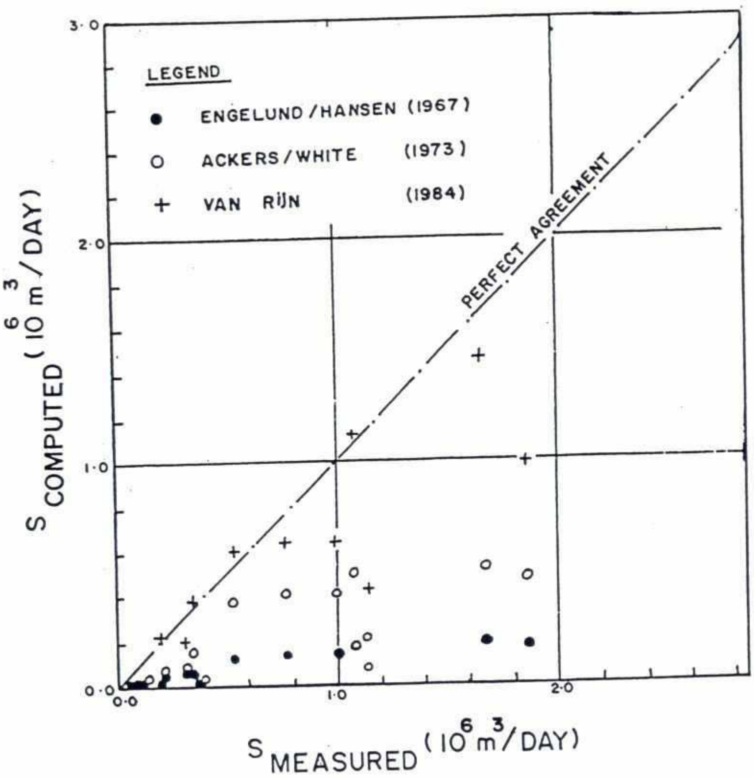


Fig. 3.3-1: Comparison of measured and computed sediment transport of the Jamuna river at Bahadurabad station
(Source : RPT, NEDECO, BCL, 1987)

Average annual sediment transport of the Jamuna at Bahadurabad station established under different studies are presented below (Table 3.3-1).

MPO Report	FEC Report	Coleman Report	Goswami Report	FAP 1 Report	CBJET Report	Present Study
0.387	0.431	0.607	0.400	0.600	0.499	0.270

Table 3.3-1: Comparison of annual sediment load of the Jamuna (10^9 tons)
(Source : CBJET, 1991)

Channel deformation in general is governed by an exchanging process between coarse suspended load and bed material. The FAP 1 study on bank material and char sediment exchange indicates that bank erosion in the study reach may add of the order of 15 to 30% to wash load (finer than d_5 or d_{10}) coming in from upstream based on an assumption that the sand fraction is probably in balance with as much material being deposited in the form of char building/bank accretion as is yielded from bank erosion. The CBJET (1991) study on size distribution of suspended load at Bahadurabad revealed that bed material load in suspended load accounts for 15-35% of d_{10} which confirms the FAP 1 study.

3.4 BED FORMS

Coleman (1969) based on his extensive study on the Jamuna river bedform data suggested four groups of bed forms depending on forms which tend to group together and to have similar ripple indexes:

Ripples : typical height 0.2 - 0.5 m;
Mega ripple : typical height 1.0 m, typical celerity 120 m/day;
Dunes : typical height 5 m, typical celerity 60 m/day, and
Sand wave : typical height 10 m, typical celerity 200 m/day.

Bedform dimensions in the Jamuna river were measured by the Jamuna Bridge Project Phase 2 study to determine a relation between bedform dimensions and flow characteristics during flood conditions of 1986 and 1987, and they also carried out hydrographic surveys between July and November 1987. Bedform dimensions were analyzed as a function of stage of flood season and a graph was made to investigate relations between dune height with water depth (Fig. 3.4-1). The maximum dune height observed was 6 m.

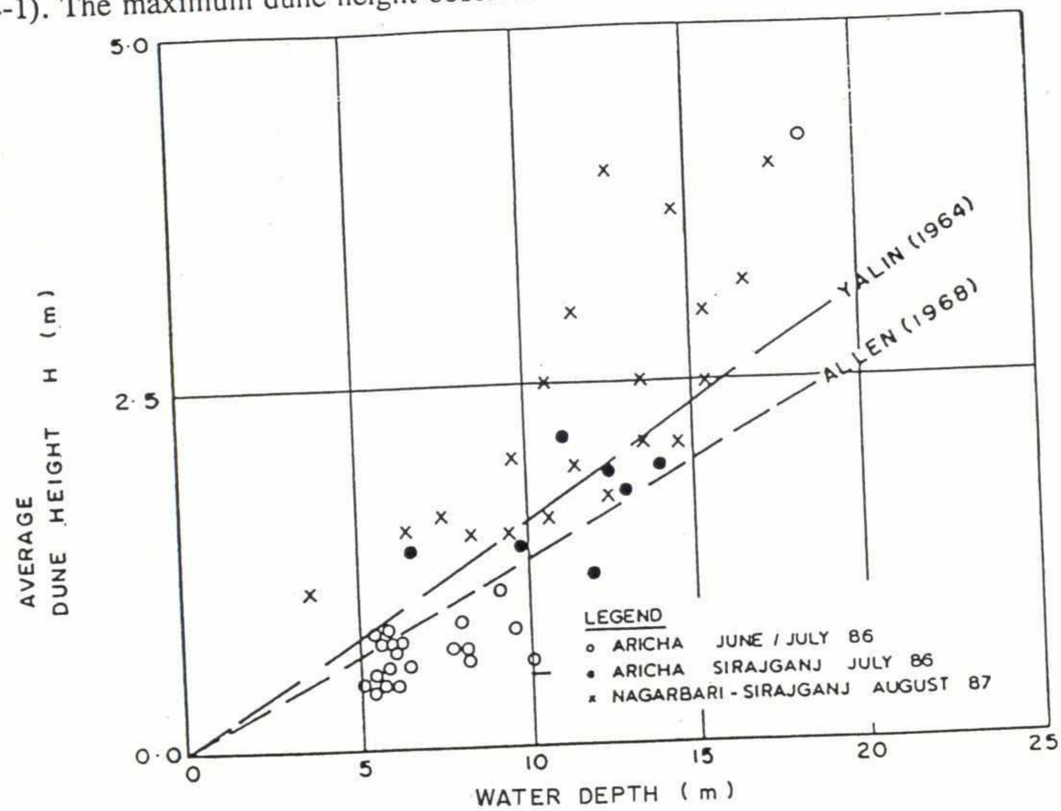


Fig. 3.4-1: Average dune height versus water depth
(Source : RPT, NEDECO, BCL, 1987)

To estimate the migration speed of the dunes, dune tracking was carried out at Sirajganj in the Jamuna river in June 1990 under the FAP 1 study. The typical height of dunes observed was 3 m. This confirms the Jamuna Bridge study observation. Both the studies differ from the maximum value of 15 m as quoted by Coleman (1969). Jamuna Bridge study states that possibly Coleman interpreted confluence scour erroneously as bed form.

3.5 BANK MATERIAL

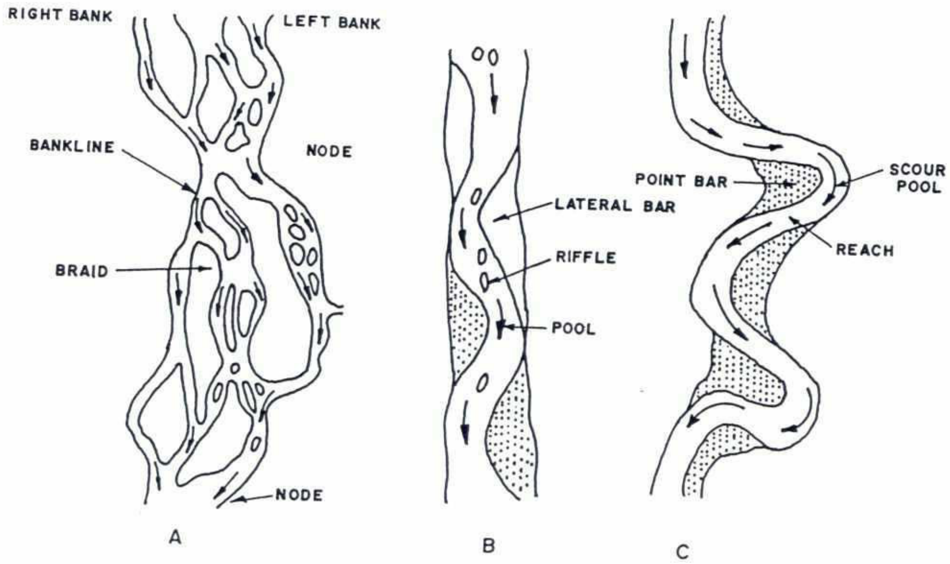
According to Coleman (1969), stratification in alluvial deposits of braided river are quite varied and complex. A study on migration of the Jamuna by Coleman (1969) has uncovered that in approximately 200 years, the river has curved a valley some 200 km long and average 13 km wide, which in other words stands for an average 18 m depth of sand displacement.

Geomorphological survey was carried out along the right bank of the Jamuna under FAP 1 study, it concludes that while distinct zones with different predominant material characteristics can be identified, ranging from almost pure sand to thin strata of cohesive material, overall the material has to be categorized as highly erodible. The consultant also feels reasonably justified to conclude that differences in bank material composition are not responsible for presence of nodal point.

4 PLANFORM

4.1 GENERAL

Rivers while draining land masses and carrying water and sediment to the sea, form and maintain an organized physical and hydraulic system. The pattern of a river is defined as its appearance in plan view and accordingly the rivers can be classified into three groups : braided, straight and meandering (Leopold et al., 1964) as presented in Fig. 4.1-1. It is common to find more than one pattern existing along the length of a river.



A. braided; B. straight; C. meandering.

Fig. 4.1-1: River channel pattern

(Source: Coleman, 1969)

4.2 PLANFORM CHARACTERISTICS

A number of studies were made on the planform characteristic of the braided Jamuna river. Most of these are based on a methodology introduced by Howard et al. (1970). Planform characteristics of a braided system can be broadly divided in:

i. characteristics of the channel network, and
ii. characteristics of the individual channel.

To characterize the channel network of a braided system braiding indices are used. Howard et al. (1970) have introduced a number topological and geometrical parameter to characterize channel network of braided rivers. Their approaches and definitions were used in the analysis of the Jamuna planform characteristics for the Jamuna Bridge Phase 1 study (for detail see G.J. Klaassen and K.Vermeer, 1988).

Klaassen and Vermeer used the following parameters in determining the braiding indices for the Jamuna river in Bangladesh :

- E average number of segments bisected by the cross-lines 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, and
- C average width of the stream between the outermost segments within the section.

Based on available Landsat images of the Jamuna river in Bangladesh, Klaassen and Vermeer for their analysis divided the river into six reaches, each about 30 km in length. Derived braiding indices of the Jamuna river for the years 1973, 1978, 1981 and 1987 for the parameters defined above are presented in Table 4.2-1.

No data were available for the upper reaches in the years 1973 and 1978, as no satellite image was available.

Date		21 Feb., 1973				22 Feb., 1978				27 Dec., 1981				27 Feb., 1987			
Parameter		E	E_1	N	C	E	E_1	N	C	E	E_1	N	C	E	E_1	N	C
		(-)	(-)	(-)	(km)	(-)	(-)	(-)	(km)	(-)	(-)	(-)	(km)	(-)	(-)	(-)	(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	22	10.0
	4	2.8	1.8	12	10.5	2.8	1.8	16	8.0	2.9	1.9	12	10.5	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 4.2-1: Braiding indices of the Jamuna river over the period 1973-1987.

(Source: RPT, NEDECO, BCL, 1987)

The following observations were made by Klaassen and Vermeer regarding the braiding indices provided in Table 4.2-1:

- 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 Struiksma 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 the reach just upstream of Sirajganj has experienced substantial changes. The 1986 data (not included in the table) support the change in braiding characteristics.

Braiding intensity on a reach by reach basis for the Brahmaputra/Jamuna river was studied by a BUET team and independently checked under the FAP 1 study. Braiding intensities, number of anabranches, and overall braiding belt width were analyzed for the same years (1973, 1978, 1981 & 1987) using the methodology introduced by Howard et al. The FAP 1 study concludes that the braiding intensity upstream of Sirajganj is tending to increase as is the number of anabranches, segments and the overall width. Downstream of Sirajganj the situation is different where the number of channels and the overall width decreases. The observations are in line with the Jamuna Bridge study.

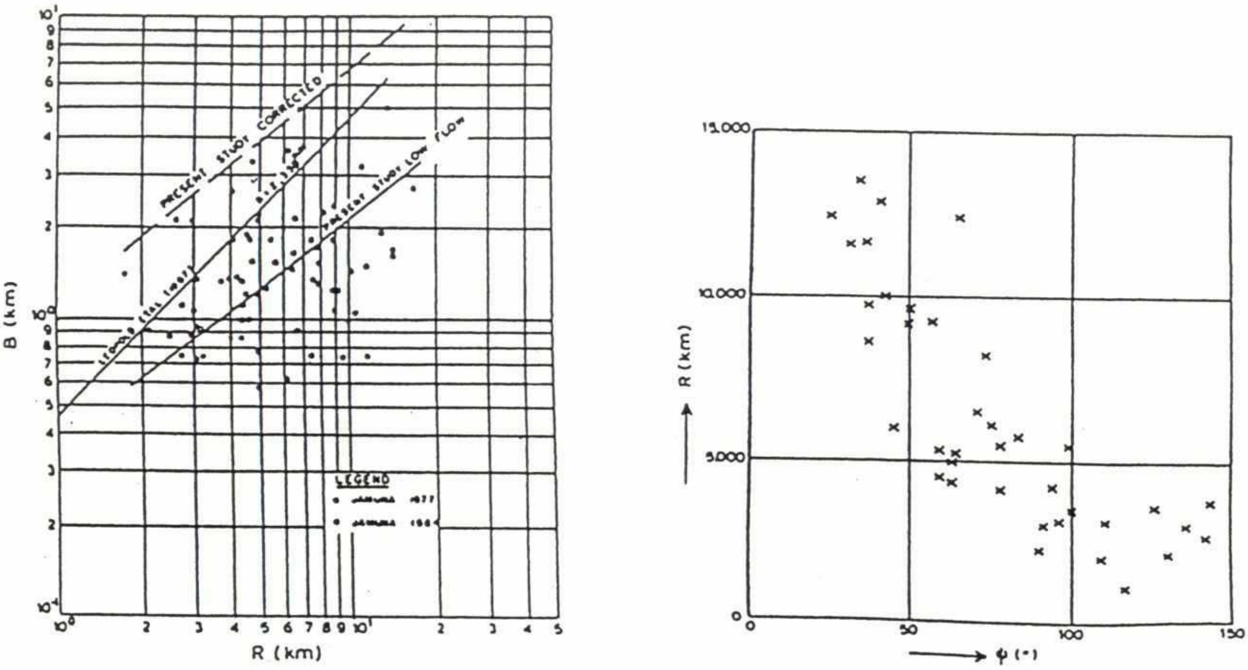
Individual channel planform characteristics are related to radius of curvature of bends (R) in relation to channel width (B), total arc length of bends (Ψ) and channel direction. Klaassen and Vermeer studied the characteristics of bends in individual channels for the Jamuna, being part of the braided network, using the Landsat images. Relation between radius of curvature, channel width and arc length is presented in Fig. 4.2-1. The plottings were based on low flow data.

In plotting the figures a relation derived from Leopold et al. (1964) for meandering channels was also used. The relation reads

$$R = 2.35\ B^{0.99} \qquad (4.2\text{-}1)$$

Fig. 4.2-1-a shows a considerable scatter from that of equation (4.2-1), which may result partly at least, from the use of low flow data.

The Jamuna Bridge Appraisal study suggested that average low flow discharge in the Jamuna river are a factor 10 lower than bankfull discharge and a consistent deviation of about 3 as used in the following plotting (Fig.4.2-1-a) may well be justified. From that it may be concluded that for the present braided river the radius of curvature are on the average smaller than for meandering channels. This analysis does not take into account the possibility that the radius of curvature may change during the transition from flood to low flow stages.



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

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

Fig. 4.2-1: Bend characteristics of the Jamuna river

(Source: RPT, NEDECO, BCL, 1989)

Arc length of bends is plotted against radius of curvature for the low flow period of 1980/1984 in Fig. 4.2-1-b. The plotting shows a much better relationship and 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 (4.2-1), which does approximately hold for bankfull discharges). Hence the major channels are less curved than the smaller ones.

4.3 NODAL POINTS

Coleman (1969) compared a total of thirteen maps ranging from 1830 to 1963 to study bank line migration of the Jamuna. For the total reach of the Jamuna from Chilmari in the north to Aricha in the south, he selected thirtytwo stations approximately 5 to 8 km apart as presented in Fig. 4.3-1.

The figure illustrates that three narrow width areas or nodal points existed in the sixties in the Jamuna. The northern most one was south of Bahadurabad, middle one at Sirajganj and the lower one was near the confluence of the Brahmaputra/Jamuna with the Ganges.

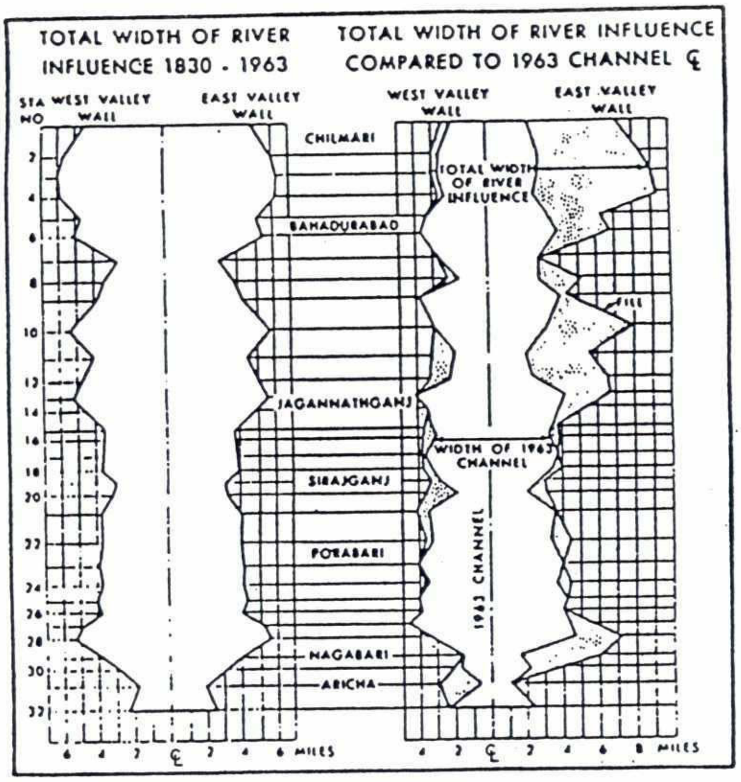


Fig. 4.3-1: Total width of the Jamuna river influence during the period 1830-1963.

(Source: Coleman, 1969)

The FAP 1 study based on their plot of retirements along the Brahmaputra right embankment and the banklines in 1956 and 1989 within the study area (from the mouth of the Teesta river in the upstream to the Bera confluence of Hurasagar) divided the river into five reaches and found respective nodal points. Based on an investigation of historical maps; 1765 (Rennell), 1830 (Wilcox), 1848-1868 updated to 1913 Survey of India maps , recent maps, Landsat and spot images, the FAP 1 study concluded that compared to relative irregular planform pattern at the turn of the century to that of 1965, the planform was beginning to show the development of a more regular pattern that is apparent today. The study observed that the majority of the nodal points were consistent with the present pattern, suggesting that these features have a reasonably long life.

In order to detect nodal points of the Brahmaputra/Jamuna river in Bangladesh, CBJET (1991) used satellite maps from 1973-1988. Seven nodes were figured in the Jamuna reach. The characteristic of nodes were found to change randomly to a certain extent each year and analysis regarding characteristic values of distance between nodes suggest that variation of channel nodes within years are not so great. Materials at the nodal points are mostly composed of alluvial soil and most of the channel nodes are not formed by a fixed boundary. These nodes, CBJET suggests that, are the products of fluvial processes with central bars and are morphologically in a strata of stability.

In the present study within the framework of the satellite study all river courses were combined and the frequency of occurrence of a deep channel at a particular location was determined. The channel pattern is seen to vary considerably both in space and time. The latest studies seem to suggest that nodal points are not as stable as previously suggested apart from the nodal point at Sirajganj. The consultant apprehends it could be well that the nodal points are related to fluvial morphological process such as presence of confluences rather than more stable reaches, a view quite similar to the CBJET study.

4.4 CHARS

The braided Brahmaputra/Jamuna river is literally choked with sand bars, locally known as chars. A river of the strength of the Jamuna, coursing its way through the extremely flat alluvial plain naturally meanders, erodes and creates new channels during the peak of its discharge. This morphological process creates a web-like configuration of channels, which are constantly migrating, the erosion destroying and the deposits recreating new islands elsewhere within the banks of the river. Char land may be defined as island chars (middle of the river), attached chars and chars mixed up with mainland (set back land) (FAP 3.1 Char study report).

The initial location and formation of the chars are determined during the flood. The Jamuna at low flow moves in quite sinuous channels seperated by numerous chars where during high floods it takes a comparative straight channel. Chien (1961) states that the channel configuration is thus destroyed or modified everytime because of the variability from low flow to flood flow and from flood flow back to low flow.

Coleman observed that chars irrespective of size move downstream almost after every flood, but in considerably varying degrees. Sometimes new chars were formed in the central channel while other chars had either been removed or drastically reduced in size.

There are some places in the middle of the Brahmaputra/Jamuna river where never a major channel has been. These are so called stable chars or mega- chars. There are large deviations in the size of the chars in the Jamuna; smallest one is less than 0.1 sq.km and the largest one being 100 sq.km (CBJET, 1991). The chars which are found to move drastically after the flood may be termed as moving chars. No detailed information is available for making fair approximation of the age of chars. The present study using series of satellite images observed that mega-chars are much smaller than previously indicated.

The CBJET (1991) study pointed out that, because of frequent shifting of the main flow of the Brahmaputra river and large amplitude of the shifting, the map showing the fluvial process of the river channel in the past 50 years demonstrate that most of the low bars are missing from time to time, and new ones are coming into being at the same time, which indicates that all except four out of six counted as stable.

It can be concluded from preliminary investigation that it takes about 5 years for the chars to develop from fairly low chars to mega-chars. It should be stressed here that this is not the typical history of chars in the Jamuna river. Most of the chars have already been eroded after a few years only. General observation is that low chars which are usually moving chars are relatively new and they may not be permanent.

4.5 BENDS

The bends of various channels or of an individual channel are determined from satellite images in an approximate way. The bend of a channel is of importance for the design of river works as it determines the scour in the outer bends, length of guide bunds and the intermediate distance between groynes and their toe levels.

The Jamuna Bridge Appraisal (JBA) study analyzed Landsat images of the Jamuna. The radii of bends of the channels, which could be distinguished against their chainage, were plotted as in Fig. 4.5-1. The following observations were made:

- Downstream of Sirajganj the radii of the channels in the Jamuna are larger than upstream.
- The radii of the Jamuna channels upstream of Sirajganj were smaller in 1977 than in 1984. In 1976 the flood was larger than in 1983, although both were above the average.

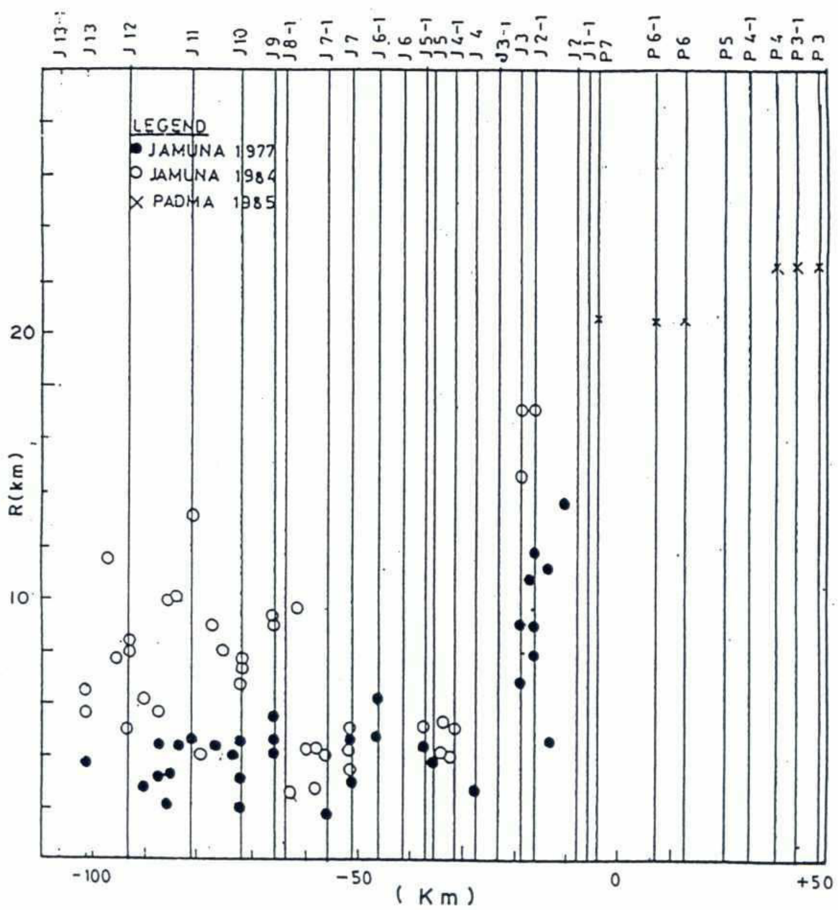


Fig. 4.5-1 : Bend radius versus chainage

(Source : RPT, NEDECO, BCL, 1987)

The width of the channels as observed from imageries were plotted against bends (Fig. 4.2-1-a) together with the relationship presented by Leopold, Coleman and Muller (1967). Although the curvature is a considerable scatter JBA concludes that the curvature increases with channel width.

The FAP 1 study gave some insight into the evolution of bends that occurs in the right bank channels of the Jamuna for the period covered by Landsat imagery. The interesting features that have emerged from their study are quoted as follows:

1. Over the period 1973 to 1990 with gaps in 1974, 1975, 1979, 1982 and 1989, 29 bends were identified as described earlier, of these 10 were concave to the right bank, 12 to the left bank and the remainder were adjoining to mid-stream chars. Although this is a small sample it may be concluded that there is approximately equal distribution of bends between both banks.

2. 6 bends concave to the right bank and 8 concave to the left bank were picked as having lives extending over at least 3 years. These represented only 60 percent and 25 percent respectively of all bends identified during the primary screening. The bend lives ranged from 3 to 7 years with a mean of 4.4 years and standard deviation of 1.2.

3. The low flow channel widths ranged from 375 to 1625 m and the radius from 1000 m to 16000 m. There was no discernible difference between the left bank and right bank in this respect.

4. There was no obvious variation in the number of bends active in any one year, although the sample is rather small for such trends to become apparent unless they are very pronounced.

5. The major active bends tend to be concentrated between Fulchari and Kazipur on the right bank and opposite Sariakandi and Bhuapur on the left bank and a scattering on both banks south of Sirajganj.

6. Of all bends analyzed, only about one quarter displayed a complete life cycle moving through steadily tightening radius until they died. It are these bends that cause most damage. Other bends were destroyed by other larger scale channels from developments before they could become fully aggressive.

7. Only two persistent bends have been identified, that are bends that have died and then reoccurred at almost the same location a few years later. These are both on the right bank, one at Sariakandi and the other one immediately north of Sariakandi. It can be noted that both are situated on the concave side and close to the apex of the macro scale change of alignment of the braided belt. The significance of this is obscure.

The FAP 1 study made specific comments on one characteristic that is of potential importance for planning embankment alignments for relatively short life horizons. They say that in most cases the aggressive bends have a relatively low ratio of lateral to longitudinal movement. This means that they typically punch into the bankline rather than shave slices off it. However there are exceptions to this rule where the bend has followed the initially lateral movement by a downstream migration and actually regenerated again in a new location.The Kazipur bend is displaying such characteristics at present.

A preliminary analysis was made on 29 well defined bends indentified on Landsat images for the period 1973 to 1980 under FAP 1 study. The study mentioned that the life of these well defined bends, which are important contributors to overall bank erosion processes are between 3 and 7 years with a mean of 4.4. Over a period of 17 years only 7 aggressive bends with a life of more than 5 years were identified.

4.6 PLANFORM CHANGES

Planform changes for a braided river are quite different to predict. Amongst other things, combination of factors responsible for changes occurring simultaneously on both the banks make it even more complicated. Studies conducted in this respect on the Jamuna river covered a number of factors like: bank erosion, channel shift, process at bifurcation and confluences, wandering of thalweg, etc. using satellite images and cross-sectional data.

An extensive analysis of all cross-sectional data has been carried out by Klaassen and Vermeer (1988). A computer programme was developed to study vertical and horizontal stability of the Jamuna in its present course. They concluded that it seems that over the last decades the Jamuna bed was stable both laterally and vertically. So the Jamuna river is on the average not aggrading, and can be supposed to be in equilibrium. This contradicts the findings of Ullah (1987), who based on a comparison of 1965/66 and 1983/84 data concluded that substantial changes have occurred.

Several studies on the Jamuna river lead to an increased understanding of the processes that take place in such large, braided sand-bed rivers (Coleman (1969), Bristow (1985), Klaassen et al. (1988)). These studies have revealed that the processes take place on a scale which can not be compared to the results of studies on "normal" braided rivers (see 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 fruitfull.

Klaassen and Masselink (1992) summarized results of related studies and presented the following characteristics about planform change of braided rivers with fine sand bed and bank material. To understand the process active for channel pattern changes in braided river systems, Klaassen and Masselink (1992) used a statical and a dynamical composite approach for the interpretation of the satellite imagery as well as the BWDB cross-sectional data.

The images are classified according to type of soil and vegetation cover and are called as statical composite. An overlay 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 the dynamical composite.

To determine bank erosion (maximum) rates for curved channels of the Jamuna river dynamical composites were used. From the study it appears that in most of the cases the value is between 0 and 500 m/year and larger values upto 1000 m/year under exceptional conditions. This confirms the findings of Coleman (1969). Some other aspects of the bank erosion were studied using the satellite images. The direction of the bank erosion was studied by plotting the direction of the maximum erosion relative to the valley slope (Fig. 4.6-1). 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.

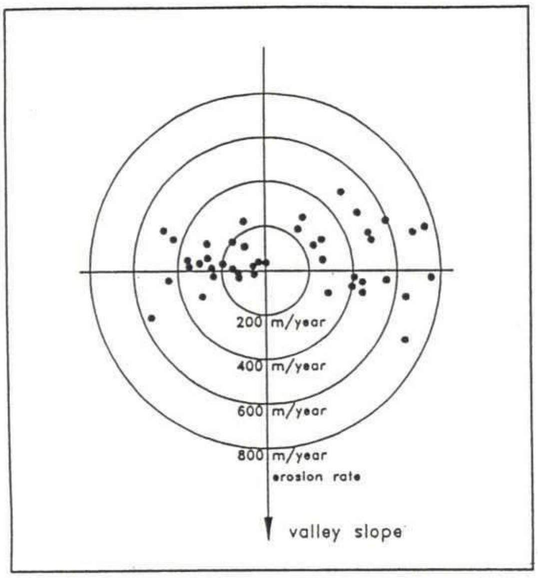


Fig.4.6-1: Direction of bank erosion along curved channels of the Jamuna river

(Source : Klaassen and Masselink, 1992)

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 edge of the channel pattern. Hence it can be concluded that for the deep channels of the Jamuna river the influence of vegetation is negligible, only effective in the upper layers (see Klaassen and Vermeer, 1988b).

Furthermore a distinction was made between the following bank erosion mechanisms: rotation (A), extension (B) and translation (C) as presented in Fig. 4.6-2.

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 noted that apparently the yearly flood hydrograph may play a role in the magnitude of the yearly erosion rate (the difference between e.g. 1976-1977 and 1984-1986).

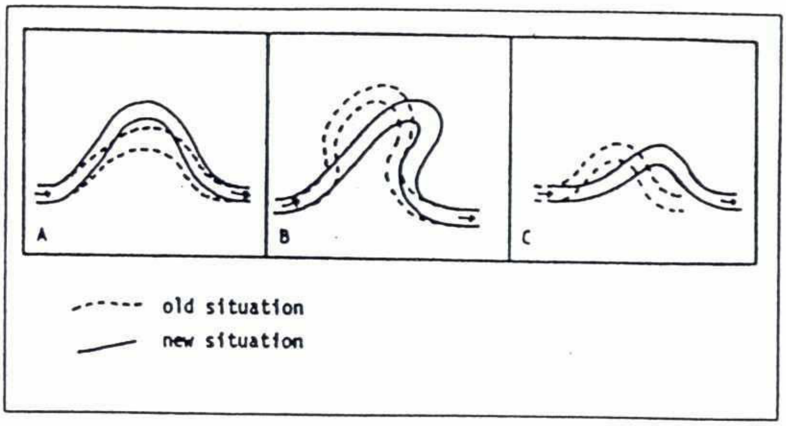


Fig. 4.6-2: Possible bank erosion mechanisms studied here

(Source : Kalaassen and Masselink, 1992)

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 :

(i) 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 as can be seen in Fig. 4.6-3.

(ii) Development of a cut-off

The development of a cut-off occurs frequently. Of the 23 cases of channel shifts studied, 11 relate to cut-offs. 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 cut-off ratio (Klaassen and van Zanten, 1989).

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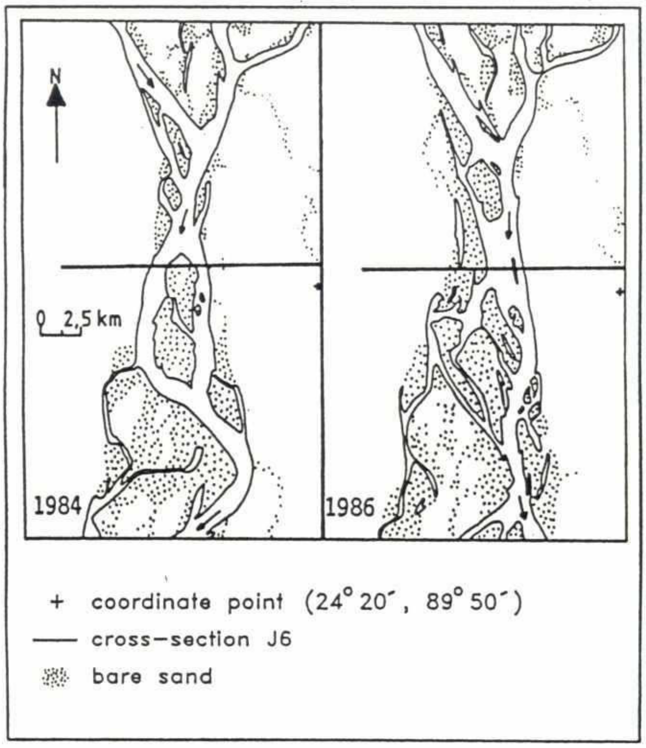

Fig. 4.6-3: Bar-induced channel shift

(Source: Klaassen and Masselink, 1992)

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.

Klaassen and van Zanten (1989) have shown the importance of the cut-off ratio λ, being the ratio between the length of the channel along the curved reach and the direct line, along which the cut-off channel is developing. For a number of cut-offs in the Jamuna river the value of λ was determined and a cumulative frequency distribution was prepared. The results are presented in Fig. 4.6-4. It appears that the values of λ varies between 1.0 and 1.7. These are very low values for the cut-off ratio, demonstrating that in this type of braided river with fine bed material channel shifting via cut-offs occurs relatively quickly. For comparison: in meandering rivers values of λ in the range of 5 to 30 have been observed (Joglekar, 1970; Klaassen and van Zanten, 1989). Furthermore it is observed that the 'critical' value of λ varies between a wide range, hence the usefulness of Fig. 4.6-4 for predictive purposes is very limited. For the time being a 50% value of 1.25 can be used.

In Fig. 4.6-4 also the frequency distribution of potential cut-off ratios of existing bends is added, and it is observed that larger (potential) values of λ apparently result almost immediately into a cut-off.

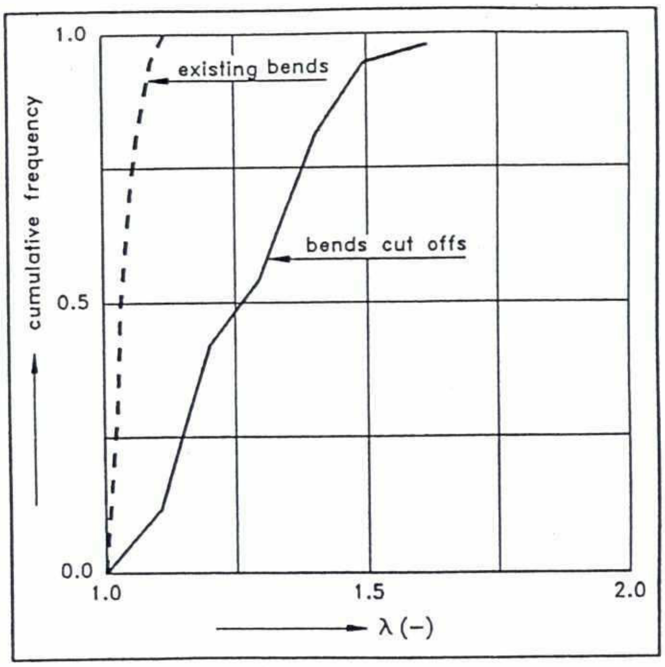

Fig. 4.6-4: Cumulative frequency distribution of cut-offs compared with existing bends

(Source : Klaassen and Masselink, 1992)

(iii) 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. In Fig. 4.6-5 an example of the development of an outer bend channel is given 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.

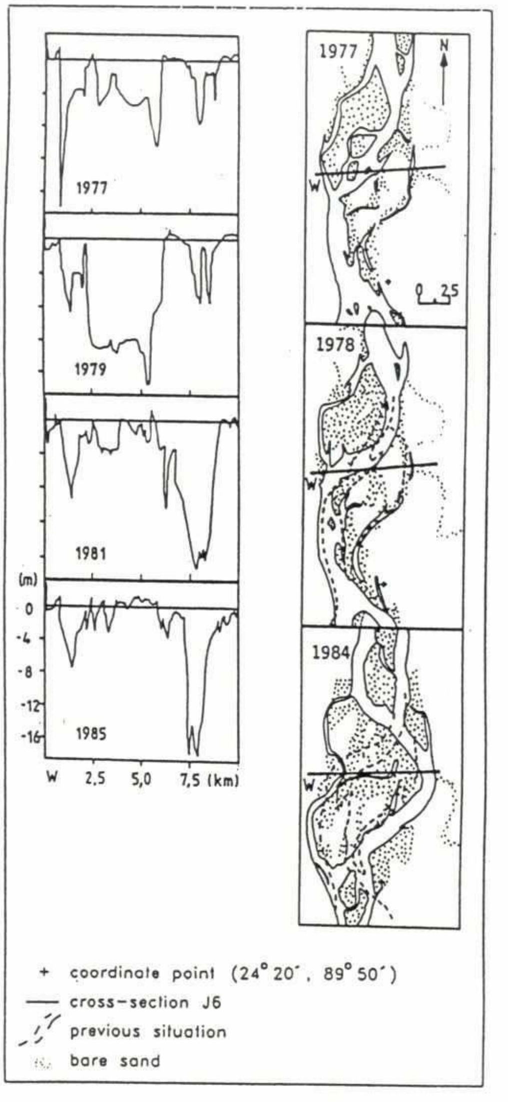

Fig. 4.6-5: Development of an outer bend channel into an already existing old channel
(Source: Klaassen and Masselink, 1992)

Processes at Bifurcation

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 separated from each other by a char; the surface of this char is above bankfull 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. 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. 4.6-6 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.

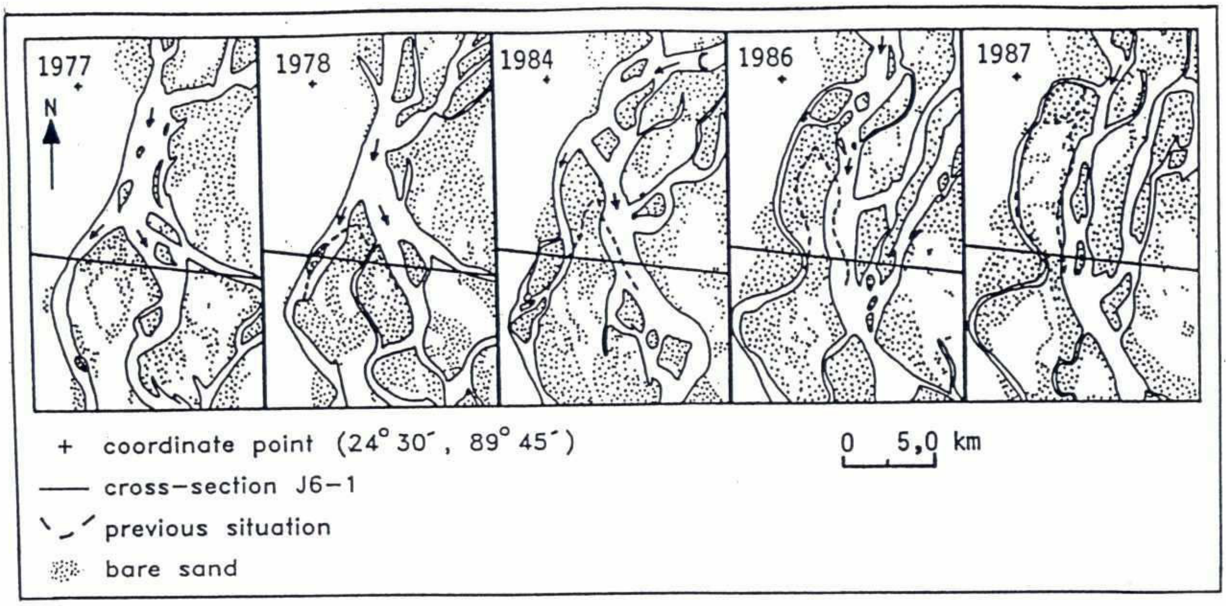

Fig. 4.6-6: Development of a bifurcation during the period 1977-1987

(Source: Klaaseen and Masselink, 1992)

Processes at Confluences

The development of a confluence appears to depend on its shape. Different shapes of confluences are schematically drawn in Fig. 4.6-7. They are different in the presence of a slackwater zone (A versus B and C) and in the angle between the two confluencing 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

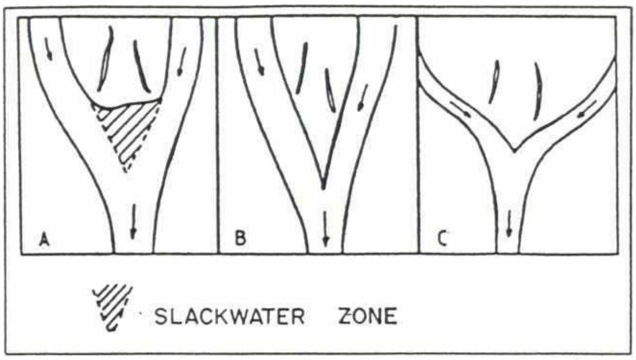

Fig. 4.6-7: Different types of confluences
(Source : Klaassen and Masselink, 1992)

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 as 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 confluencing 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.

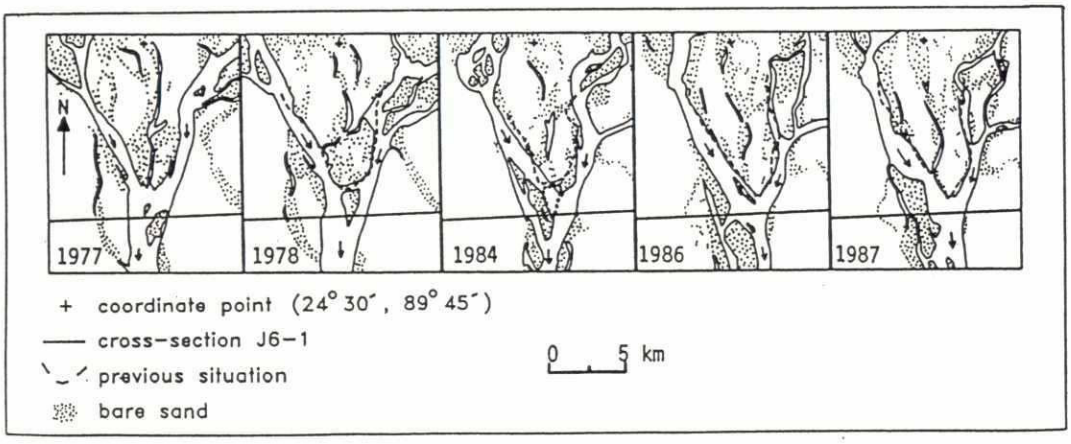

Fig. 4.6-8: Development of a confluence in the period 1977-1987
(Source : Klaassen and Masselink, 1992)

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As an example here the historical development of a confluence near Sirajganj over the period 1977-1987 is shown in Fig. 4.6-8. This confluence had a fairly blunt shape in 1977. In 1987 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 remarkably 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).

5 CHANNELS

5.1 BANKFULL DISCHARGE

In single channel rivers which are in dynamic equilibrium (that is which have alluvial mobile boundary materials but which are not aggrading, degrading or changing their width through time) the morphological expression of dominant discharge is the bankfull capacity. There are ample evidences from rivers with a wide variety of bed material types that dominant flow corresponds to bankfull flow in terms of discharge magnitude, and to a lesser extent in terms of flow frequency (Richards, 1982; Knighton, 1984). However in multi-channel or braided rivers this is thought to be case (Lee and Devies, 1986; Biedentarn et al., 1987). In fact dominant discharge is believed to be less than bankfull discharge in braided rivers (FAP 1 study).

The FAP 1 adopted a qualitative approach based on visual examination of water surface elevation corresponding to a dominant discharge of 38,000 m^3/s in relation to char top elevations at all surveyed cross-sections for the 1988/89 survey. On this basis, for the upper chars to be inundated requires 'bankfull' flow of perhaps 60,000 m^3/s, compared to JBA study estimate of 45,000 m^3/s (FAP 1 study, Annexure 4).

CBJET (1991) plotted stage discharge relation curves based on observed discharge data in 1988-89 at Bahadurabad station and water stage data at a number of stations (from Chilmari upstream to Mathura downstream) in the Brahmaputra/Jamuna river and selected 60,000 m^3/s as bankfull discharge.

The present study used 44,000 m^3/s as bankfull discharge.

5.2 CHAR LEVELS

FAP 1 studied the char over topping discharge for the Jamuna river. It was evident from water surface elevations corresponding to dominant discharge in relation to chars top elevation for river cross-section made in 1988/89 that two distinct char levels may be discerned along the course of the river. Dominant flow was found to corrrespond to char over topping discharge only for the lower chars, while for the upper chars the over topping discharge corresponds to bankfull flow, which FAP 1 estimates as 60,000 m^3/s. Fig. 5.2-1 represents mega-chars according to FAP 1.

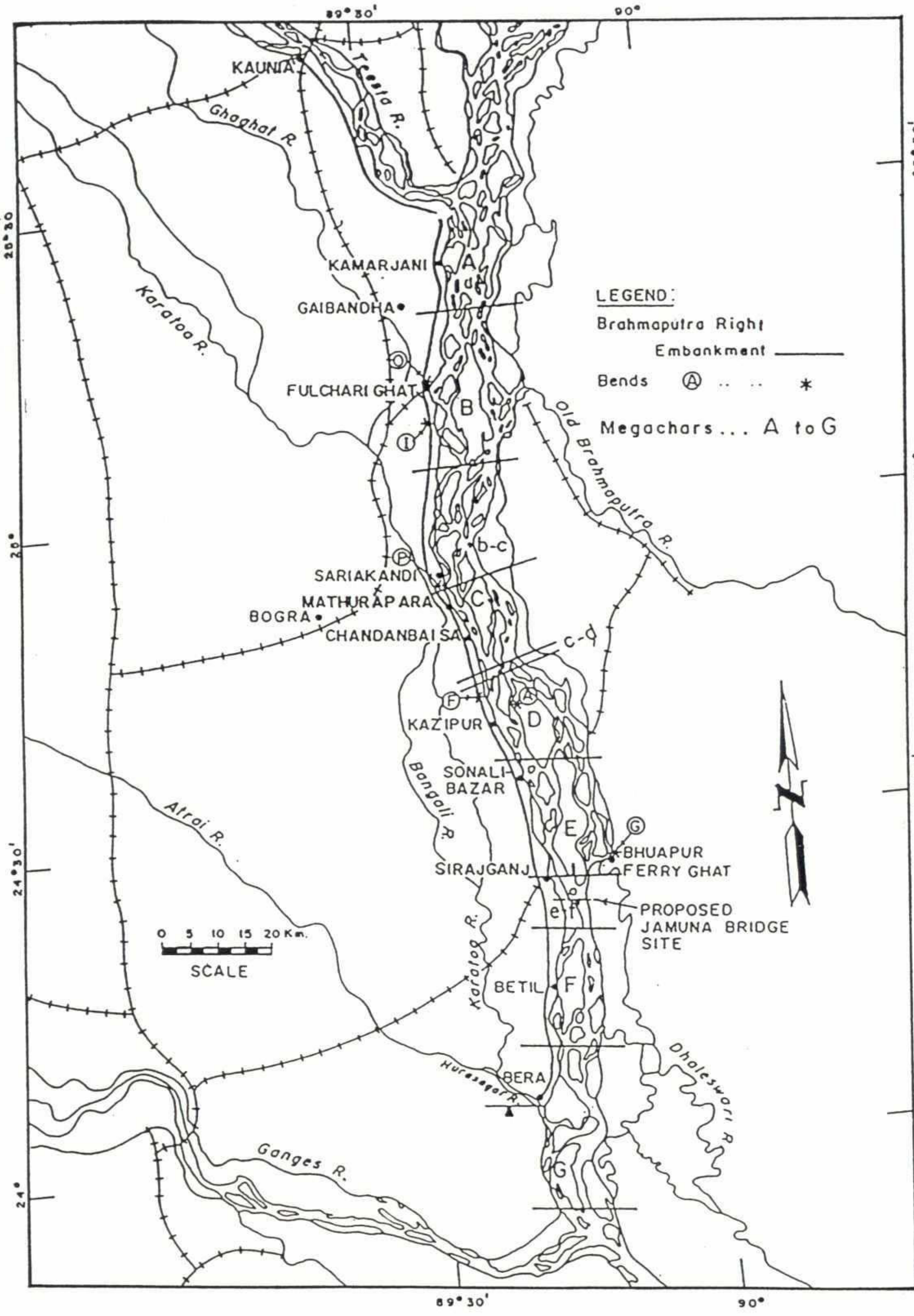

Fig. 5.2-1: Mega char according to FAP 1

(Source : HALCROW, et al., 1991)

Little is known about the formation and growth of mega-chars. Once they are above a certain level, and assuming that they are not eaten away by eroding bends, vertical accretion will take place. The higher the char level is, the finer will the particles be that will settle on the char surface. Hence it may be expected that gradually also finer silt and clay particles will be deposited. This will allow vegetation on the chars, and this in return will accelerate the process of vertical accretion. At present some of the mega-chars are at approximately the same level as the floodplain, and hence they are inundated only during floods above bankfull stage. The accretion of fine clay particles improves the quality of the soils and encourages people to try to settle on the chars.

It was attempted to determine the celerity of this vertical accretion process. This was done by plotting the level of char land as can be determined from the latest cross-sections available from BWDB (and relating to the 1989-1990 low flow season) versus the age of the chars. The result is presented here as Fig. 5.2-2.

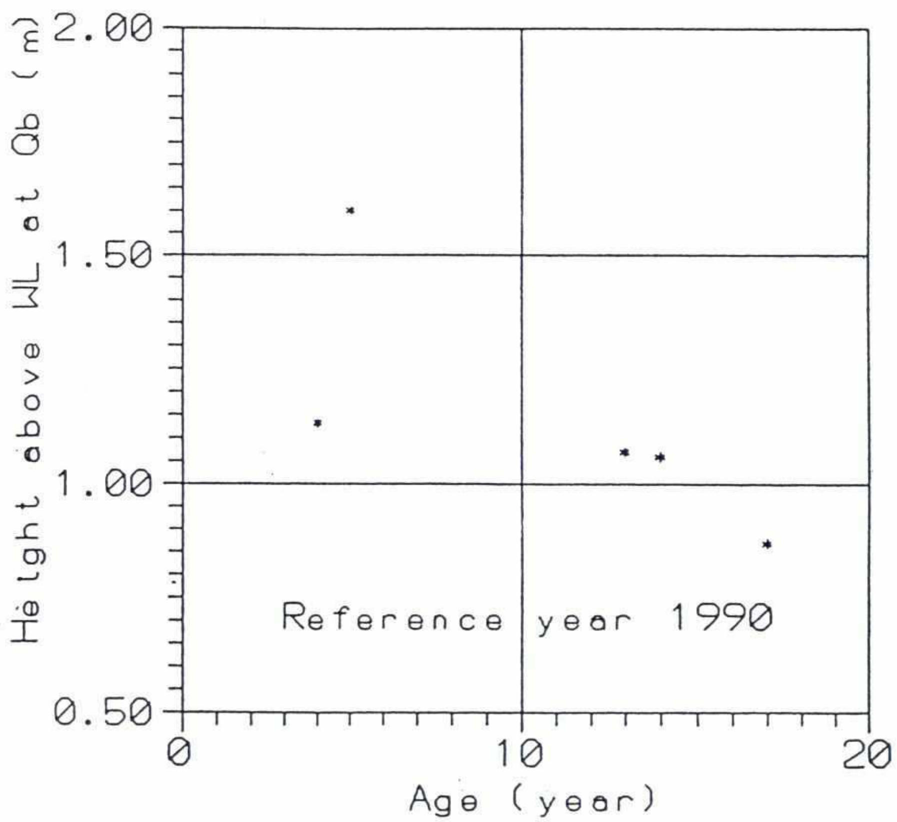

Fig. 5.2-2: Level of chars versus their age

It is assumed that a similar process of vertical accretion is applicable to attached bars. Hence it takes quite some time before these mega-chars and attached bars can be used for more permanent housing and cultivation.

From a careful look into vertical accretion process of char building, it can be concluded that the level of mega-chars which are permanent does not change significantly after 4 or 5 years.

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5.3 CHANNEL DIMENSIONS

The Jamuna Bridge Phase 2 study dealt in detail with equations for determining the hydraulic geometery of the Jamuna river, which also the present study accounts as considerably reasonable.

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 :

$$p = 2.67\, Q^{1/2} \tag{5.3-1}$$

$$R = 0.47 \left[\frac{Q}{f}\right]^{1/3} \tag{5.3-2}$$

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

$$f = 1.59\, D_{50}^{1/2} \tag{5.3-3}$$

where D_{50} = diameter of the bed material 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 :

$$p = 3.3\, Q^{0.512} \tag{5.3-4}$$

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 braided Jamuna river to these regime equations. This was done for bankfull discharge (44,000 m^3/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 Fig. 5.3-1. The plotted data can be described by the following relations :

$$\bar{h} = 0.23\, Q^{0.32} \tag{5.3-5}$$
$$B = 16.1\, Q^{0.53} \tag{5.3-6}$$

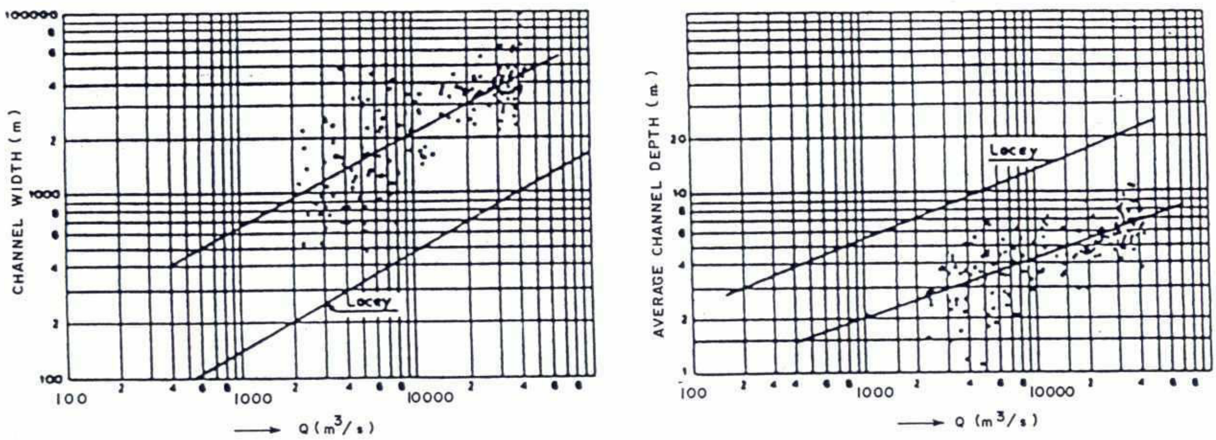
Both relations are in S.I - units. The Lacey equations, expressed in these units and assuming p = B and R = $\bar{h}$ for these very wide channels, read:

$$\bar{h} = 0.47 \left[\frac{Q}{f} \right]^{1/3} \qquad (5.3\text{-}7)$$

$$B = 4.81\ Q^{1/2} \qquad (5.3\text{-}8)$$

with D_{50} = 0.2 mm and f = 1.13, the equation (5.3-7) can be written as :

$$\bar{h} = 0.45\ Q^{1/3} \qquad (5.3\text{-}9)$$



(a) width versus discharge (b) average depth versus discharge

Fig. 5.3-1: Regime relationship of the Jamuna river channel at bankfull stage
(Source: RPT, NEDECO, BCL, 1989)

Comparison of the equations (5.3-8) and (5.3-9) to the equations (5.3-5) and (5.3-6) yields the following conclusions:

- the exponent of the discharge is approximately the same in both the comparable equations;
- the Jamuna channels are substantially wider and shallower 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 Fig. 5.3-1 is at least partly due to the channel direction not being perpendicular to the cross-section. However, also other factors will play a role.

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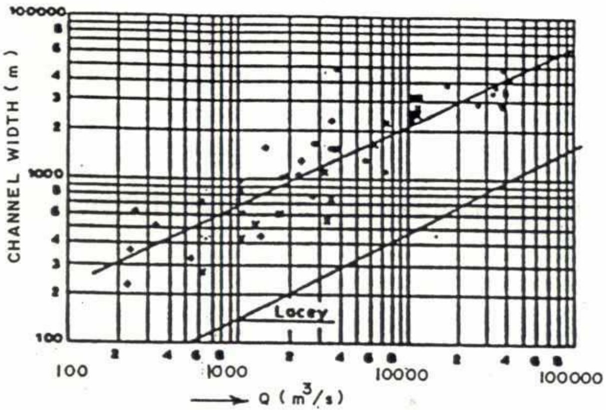
(ii) Lacey proposed a third equation for the slope of a stable channel. Introducing the relevant figures in this equation results in a slope which is 20% smaller than the actual slope of the Jamuna river.

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 Fig. 5.3-2). Under this assumption the average depth and width are described by :

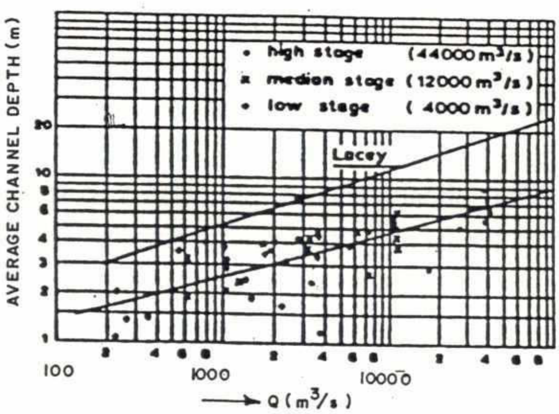
$$\bar{h} = 0.56\ Q^{0.23} \qquad (5.3\text{-}10)$$

$$B = 18.9\ Q^{0.51} \qquad (5.3\text{-}11)$$

It is remarkable how similar these equations are to the equations which relate to bankfull discharge only: equations (5.3-5) and (5.3-6). Apparently the Jamuna channels can to some extent be considered as regime channels for lower stages also, possibly indicating that morphological processes in the Jamuna river are relatively fast.



(a) width versus discharge



(b) average depth versus discharge

Fig. 5.3-2: Average at-a-station relationship for the Jamuna river channels

(Source: RPT, NEDECO, BCL, 1989)

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6 SCOUR

6.1 INTRODUCTION

Scouring of the river bed in alluvial rivers often endangers the stability of structures in the river or along the river bank. Different types of scour are distinguishe as : general scour (section 6.2), bend scour (section 6.3), confluence scour (section 6.4) and local scour (section 6.5). The most important parameter of scouring is the maximum depth of the scour hole. In this chapter the most important formulas for an estimation of the scour parameters are summarized. Most of these formulas include emperical coefficients and in the validity range of these formulas the accuracy of the scour depth estimation is 20 to 30 %. Therefore the local scour holes near the bank protection structures which are designed in this project, are investigated in a physical model. The results of that model investigation are described in Section 8.7 of Volume A of the Main Report and in ANNEX 14. Some other types of scour can be estimated and studied with a mathematical model.

6.2 GENERAL SCOUR

Bed levels in alluvial rivers vary with time and place due to many causes. Some of these are of local nature and are of short duration. However, long term morphological development is caused by changes in boundary conditions of the river. Long term morphological scour is usually termed as general scour. This is the overall lowering of the bed level owing to changes and developments in catchment area.

Two phenomene closely related to scouring are so complex that precludes efforts to fully understand the scouring processes, they are:

- turbulence, with its great complexity and variability, and
- sediment transport with its strong dependence on the interaction with turbulent flow.

A number of studies were carried out to have an overall comprehension of channel deformation of the rivers in Bangladesh.

The Jamuna Bridge Phase 2 study developed a one-dimensional mathematical model (RIVMOR) to simulate long term aggredation and degradation processes in rivers. The model was based on a rough schematization of the Brahmaputra/ Jamuna river from Dibrugarh (India) to the confluence with the Ganges river.

The boundary conditions in the complete model were adjusted such that the model was in equilibrium with certain assumed developments. The consequences of all the scenarios for the stages and bed levels near Sirajganj were simulated with the mathematical model for a period of 75 years. The following Table 6.2-1 presents the changes in bed level and water level (during dominant discharge) at Sirajganj (for details see Jamuna Bridge study, Design Report).

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rise (+) and fall (-)

Condition	Bed Level	Water Level
Realistic Situation	+ 0.5	+ 0.5
Maximum Scour	- 0.2	- 0.2
Maximum Aggradation	+ 1.7	+ 1.2

Table 6.2-1: Change of bed level and water level at Sirajganj

(Source: RPT, NEDECO, BCL, 1989)

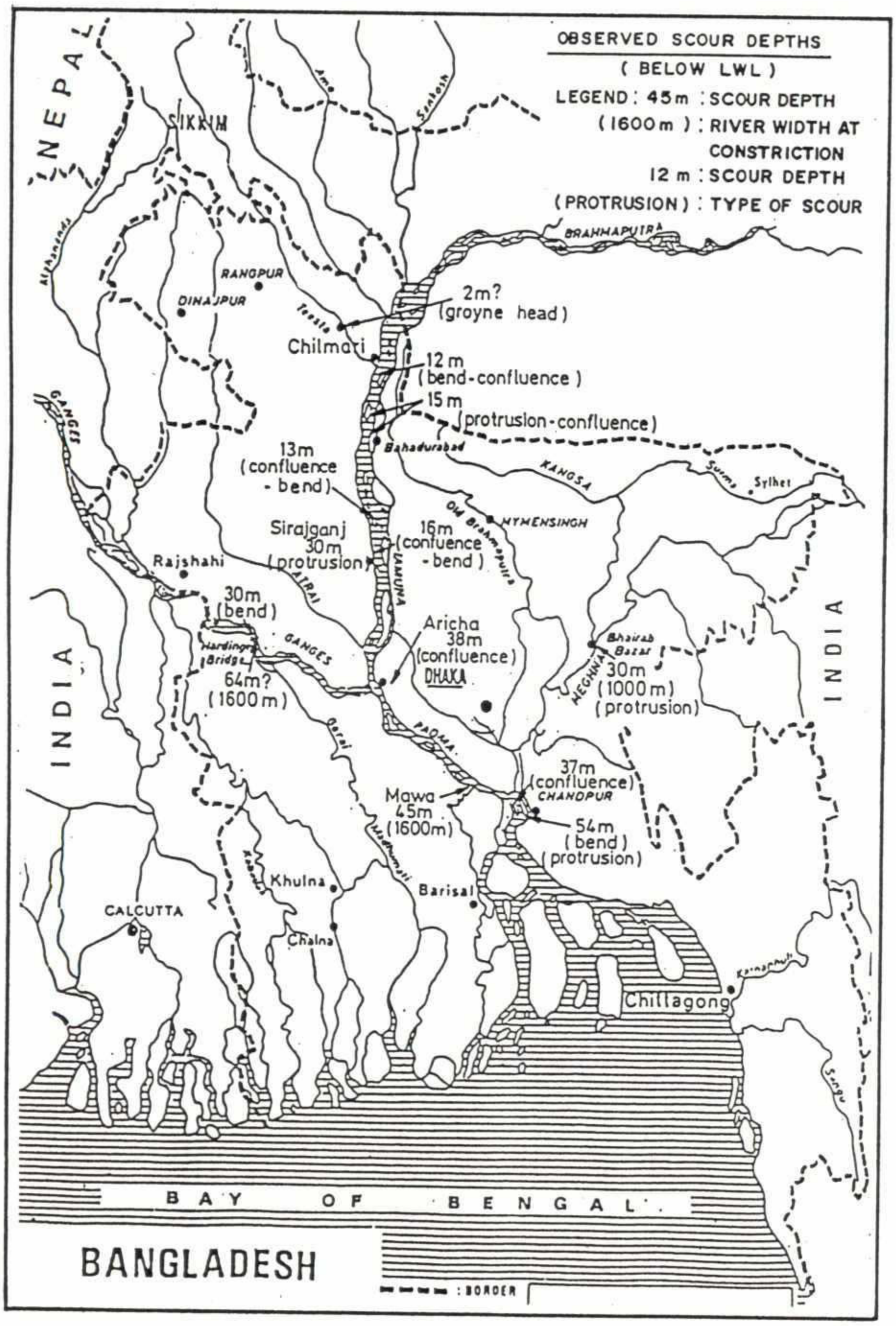


Fig. 6.2-1: Observed scour depths in Bangladesh

(Source: FEC, et al., 1989)

A pre-feasibility study for flood control in Bangladesh conducted by the French Engineering Consortium (FEC), BWDB and BETS in 1989 suggests that a rough estimate of 1 m for general aggradation for rivers in Bangladesh is fairly reasonable. Fig. 6.2-1 shows the scour depth observed in major rivers in Bangladesh, together with their type of location (FEC, 1989). It is important to note that most of these scour depths are related to highly exposed points related to structures; for instance, bridges, large confluences and accidentally submerged spurs/dykes.

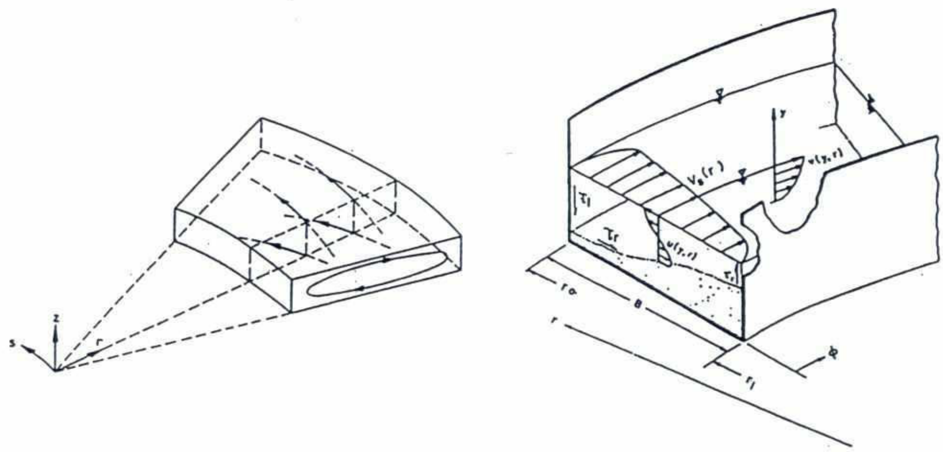
According to the CBJET (1991) report the amplitude of annual variation in the Brahmaputra/ Jamuna river bed in aggradation and degradation is generally between 0.5 - 1.0 m, which is conform to other study results.

6.3 BEND SCOUR

The water surface in a bend rises towards the outside in accordance with the equation of motion and tends to scour more deeply owing to spiral flow created there across a bend. Neither the water surface nor the bed elevation are consistent. They depend not only on the nature of the bend (i.e. plane shape, bank erodability and segregation to bed material) but also on lateral water surface slopes, spiral flow etc.

For axially symmetric flow in a bend, a lateral pressure gradient or a lateral slope is generated, which is equal for all the water particles in a vertical line. But the water particles near the surface will have a higher velocity than the average velocity, whereas those near the bed, will experience a velocity less than the average. Thus particles in a bend follow a curve of variable radius causing a helical flow pattern, commonly known as spiral flow.

The following Fig. 6.3-1 shows the formation of helical flow in a 3-D sketch of the velocity distribution in a bend.



Helical flow in river bend 3D-sketch of velocity distribution in bend

Fig. 6.3-1: Formation of helical flow and velocity distribution in a bend

(Source : Mesbahi, 1992)

This spiral flow has a large influence on the bed profile as it transports sediment from the outer bend towards the inner bend until a lateral slope is formed and an equilibrium between lateral components of bed shear stress forces and gravity forces is obtained (Van Bendegom, 1947).

In an alluvial channel bend with dominant bed load, the cross-sectional profile can be described by (Struiksma et al. (1985)):

$$\frac{h}{h_c} = \left[\ 1+0.5\ .\ A\ .\ 0.85\sqrt{\theta}\,(1-R/R_c)\right]^{-2} \qquad (6.3\text{-}1)$$

in which h = water depth (m), R = radius of curvature (m), A is a factor (usually between 5 and 10) indicating the influence of the transverse bed slope on the bed load sediment transport (-), and θ = Shields parameter (-). The index c refers to the channel axis (see also Jamuna Bridge Phase 1 report, Appendix C.3).

The above equation (6.3-1) holds good for axi- symmetric conditions and is only applicable for some limiting conditions, notably:

(i) "equilibrium" profile, that develops only at a considerable distance downstream of the beginning of the bend;
(ii) sufficient time has elapsed for the bend scour to develop fully, and
(iii) suspended load is negligible.

With the present understanding that a large depth in an outer bend is expected to cause a considerable contribution to maximum scour depth, the Jamuna Bridge study carried out hydrographic measurements in a number of selected reaches to make a fair prediction of maximum bend scour during extreme flood conditions. Measurements were repeated to observe the scour during the flood season and the subsequent period. The results of the repeated measurements of bend scour are presented below (Table 6.3-1).

River reach	Data	Maximum bend scour (m + PWD)
Pechkaholo	17 Aug 1987	- 1.5
	21/22 Nov 1987	+ 0.3
Bhuapur	24/25 Aug 1987	- 2.4
	23 Oct 1987	- 2.1
Pingna	26/27 Sept 1987	0.9
	29/30 Oct 1987	0.9
Dinghapara (two bends)	11/13 Nov 1987	+ 5.5 /+ 1.8

Table 6.3-1: Repeated measurement of bend scour in the Jamuna

(Source: RPT, NEDECO, BCL, 1987)

Examples of bend profiles at a certain bend are shown in Fig. 6.3-2.

Based on their observations the Jamuna Bridge study states that the maximum bend scour in the surveyed bends appeared at the down stream end of an identified channel bend and that the maximum scour was less than the theoretical equilibrium scour.

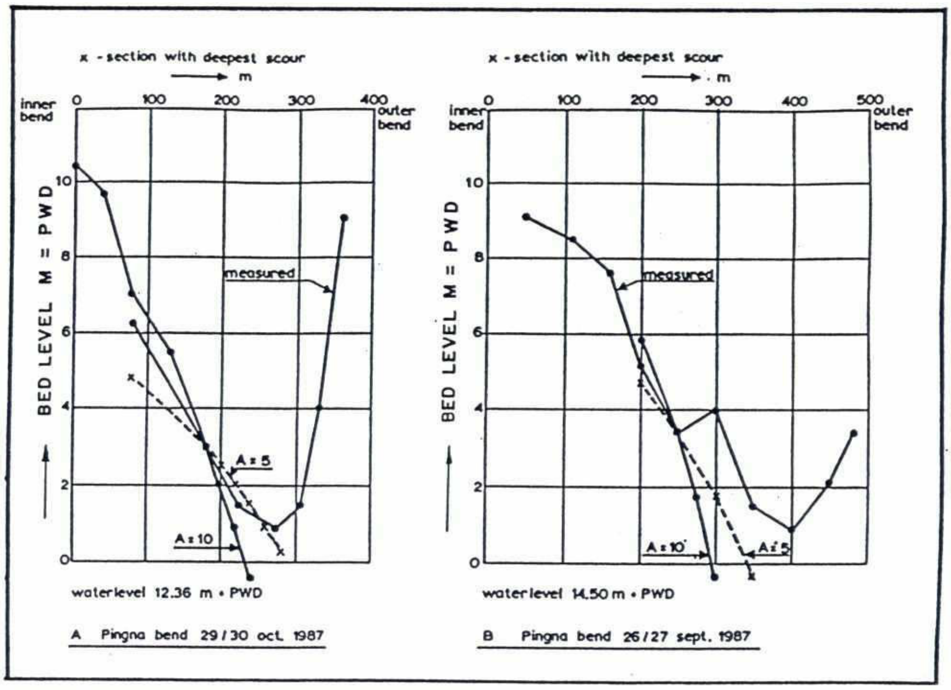


Fig. 6.3-2: Bend profile at Pingna compared with theoretical bend profile
(Source: RPT, NEDECO, BCL, 1987)

Therefore, it seems justified to assume that bend scour is not increasing due to additional input of bank erosion products. The bend scour is expected to increase again once the bank protection works are implemented.

Two-dimensional mathematical modelling of the flow and morphology of a river bend is now-a-days possible. Using the flow model developed by De Vriend (1977, 1981) and Struiksma et al. (1985) the bed topography in bends with fixed side walls can be computed. Results agree well with experiments in a laboratory flume.

6.4 CONFLUENCE SCOUR

If two streams come together or bifurcated channels reunite, they do so at an angle and at variable levels. The resulting spiral flow motion can be effective in producing a region of significant scour, termed as confluence scour. Very little information is available about confluence scour in literature. A methodology developed by Ashmore and Parker (1983) to predict maximum confluence scour of gravel bed rivers with coarse material was adopted by the Jamuna Bridge study to expand the method to large sand bed river.

The Jamuna Bridge study proposed a relationship between confluence scour (h_s), approach channel depths (h) and approach channel convergence angle (θ), which reads

$$\frac{h_s}{h} = 1.292 + 0.037 \; . \; \theta \qquad (6.4\text{-}1)$$

where $h = (h_1 + h_2)/2$, and h_1 and h_2 are the depths in the two approach channels.

This relationship was tested using data from Sirajganj and from special hydrographic surveys conducted in 1986/1987. The results indicated that observed and predicted scour depths vary in the order of $\pm$ 20%.

6.5 LOCAL SCOUR

In general local scour is caused by the man-made structures, which are built in the river, such as dikes, guide bunds, groynes, bridge piers, abutments, concentrate the river flow and can therefore cause local scour.

The interesting parameters of local scour are :
- Maximum scour depth
- Location of maximum scour depth
- Side slope of scour hole

The maximum scour depth according to different past studies is affected by all or some of the following parameters :
- the size of material in the river bed
- the dominant discharge
- the discharge per unit width of channel (at dominant discharge)
- the upstream mean water depth in the channel
- the upstream Froude number
- the sediment load of the flow

and for river training works also :
- the inclination of spur-dykes
- the slope and shape of groyne heads

The estimate of the maximum scour depth around groynes according to regime approach is summarised in Breusers and Raudkivi (1991), the relevant part of their manual is quoted hereafter :

Inglis (1949) analyzed field data on the maximum scour depth observed near spur dikes and guide bunds in India and Pakistan.He compared the total scoured depth, $y_o + y_s$, (y_o being initial water depth, y_s scour depth) with the three-dimensional Lacey regime depth, y_{3r}, which can be obtained from the equation:

$$y_{3r} = 0.47 \left[\frac{Q}{f}\right]^{1/3} \tag{6.5-1}$$

where Q = flood discharge, f = silt factor, is equal to $1.76\sqrt{d}$ (d is average bed material size in mm).

The ratio $(y_o + y_s)/y_{3r}$ ranged from 1.6 to 3.9, and Inglis (1949) recommended the use of the following values :

- Scour at straight spur dikes / groynes with steeply sloping noses	3.8
- Scour at similar dikes but with long sloping noses	2.25
- Scour at guide bank noses of large-radius	2.75

Ratios over the observed range should be used with judgement as to the severity of the river's attack on the structure. These high values were observed for 'single groynes' which protruded far into the river and are therefore severely attacked. Scour depth will be less for a well designed system of groynes.

Laboratory studies of these structure were performed in flumes with fixed vertical side walls and erodible beds, and they could therefore be compared with Lacey's two-dimensional regime depth, y_{2r},.

defined via :

$$y_{2r} = 1.34 \left[\frac{q_1^2}{f}\right]^{1/3} \tag{6.5-2}$$

in which q_1 is the discharge per unit width in the contracted section.

The most readily useful of the available studies are those of Ahmad (1953) which provided, particularly, information on the effect of the angle α on the depth of scour, and of Liu et al. (1961) whose results cover the widest range of the pertinent variables. Most of the available results are for spur dikes in the form of a vertical wall. Various other studies have added marginally to the limited information on this subject: Laursen and Toch (1956), Laursen (1958, 1963), Field (1971), Veiga da Cunha (1971), Karaki et al. (1974) and Richardson et al. (1975). In a related study Nwachukwa and Rajaratnam (1980) observed dramatic increases in boundary shear stress close to the end of a spur dike.

Ahmad presented his results relating scour to the regime depth on the basis of an equation with the form

$$y_o + y_s = K q_1^{2/3} \tag{6.5-3}$$

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which is compatible with the Lacey regime equation (K taking the place of $1.34/f^{1/3}$). His results for the effect of angle are summarized, with other relevant factors are presented hereafter.

Recommendation for design :

Scour around spur dike

Satisfactory results can be obtained from empirically determind values of K in the equation (6.5-3), with value of 2.0±15% for a nearly vertical spur dike. Correction factors modify this result for various other conditions as follows :

Spur dike angle α

α	30°	45°	60°	90°	120°	150°
K_1	0.8	0.9	0.95	1.0	1.05	1.1

Shape of dike	K_2
Vertical board	1.0
Narrow vertical wall	1.0
Wall with 4.5° side slopes	0.85

Position of dike	K_3
Straight channel	1.0
Concave side of bend	1.1
Convex side of bend	0.8
Downstream part of bend, concave side	
Sharp bend	1.4
Moderate bend	1.1

To an acceptable approximation, the combined use of the various factors (K, K_1, K_2, K_3) is recommended.

Scour near Guide bund

The Jamuna Bridge Phase 2 study carried out a model test to determine the local scour near a guide bund. The local scour was determined through the formula :

h_s	=	$h_{int} + h_r \times (k_3-1)$,
h_s	=	scour depth (below water level),
h_{init}	=	initial depth at "commencement" of the local scour,
h_r	=	"regime" depth, for the guide bund case determined at 18m (assuming $Q = 35000\text{-}4000\ m^3/s$),
k_3	=	model factor, (as obtained from model tests and literature mu k_3 = 2.2, sigma k_3 = 0.12).

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For detail about scour around guide bund, reference is made to the Jamuna Bridge Phase 2 study, Design Report, River Training Works, Vol. II.

Scour Around Revetment

Along revetments local scour holes do occur, even if the velocities are parallel to the revetment. This is probably due to (i) the decrease in depth going towards the river bank, and (ii) the difference in roughness of the revetment and the adjacent river bed, both phenomena producing horizontal eddies. These eddies in turn cause local scour. No specific design formula are available for local scour along revetments.

The Jamuna Bridge Phase 2 study carried out a scale model investigation of the existing revetment of Sirajganj. It was found that the scour depth was the same as the water depth, or :

$$h_s = \alpha\ h_o \text{ with } \alpha = 0.3 \qquad (6.5\text{-}4)$$

As the slopes of the bank protection works to be designed will have slopes of 1V:3.5H, while the slopes at Sirajganj were about 1V:2H, it seems fair to reduce this value (see also the reduction factor K_2 given earlier).

Scour around groynes

For the local scour around groynes the expression suggested by Ahmad (1953), as cited in Breusers and Raudkivi (1991), can be taken. These are given in regime approach discussed earlier. It seems logical to use for K_3 a value of 1.0, as the effect of the outer bend scour that develops in the outer part of river bend is taken into account.

So long the discussion on local scour dealt with some formulas to predict the maximum scour depth but only with a low level of accuracy (30 to 50%) and no information to determine the size of the scour hole. Local scour is important for design of the falling apron and the soil mechanical stability of the whole structure. For the design of a falling apron, the maximum scour depth (of which local scour is a dominant contributor) and the shape of the scour hole are important parameters. Therefore the study conducted physical model tests to asses all important parameters related to local scour near to groynes and revetments. Moreover, no information is found in the literature on the local scour holes near the permeable groynes.

6.6 COMBINATION OF DIFFERENT TYPES OF SCOUR

In nature combinations of different types of scour can be observed. For example: if groynes are placed in an outer bend the local scour hole should be combined with the bend scour profile, if in a degradating river groynes or a revetment are designed the local scour depth should be increased with the depth of the general scour, and if near the head of the groyne two channels join a risk of a deep scour hole as a combination of the confluence scour hole and the local scour hole of the groyne exists.

For these type of combinations no validated formulas can be found in the literature. Therefore the combination of bend scour and local scour scour near a groyne head is investigated in a physical model, see ANNEX 15.

For the combination of general scour and local scour the estimated maximum scour depths can be added to estimate the final maximum depth of the local scour hole. Because the length over which the river degradates is much larger than the size of the local scour hole.

For the combination of the local scour hole around a pile of a surface screen and the local scour hole under that screen the local scour hole of the pile can often be added to the local scour hole under the screen as a first estimation. This is valid if the diameter of the pile is small compared with the local scour hole under the screen.

For the combination of a confluence scour hole with a local scour hole no general rules exist, because this combination is more complicated.

7 CHARACTERISTICS OF OTHER MAJOR RIVERS

7.1 INTRODUCTION

Bangladesh is situated in the flood plain of three great rivers, the Brahmaputra, the Ganges and the Meghna. These major rivers drain a total catchment area of about 1.72 million sq.km of which only 7 percent lies within Bangladesh territory. The total stream flow with huge sediment load originating in the upstream catchment in India, Bhutan and China passes through the country. Most of the country is quite flat with half of the land area situated at a level lower than 12.5 m above Mean Sea Level.

Of the three major rivers passing through Bangladesh the Brahmaputra/Jamuna plays a key role during floods in Bangladesh, and the present study concentrated on this river. The Brahmaputra/Jamuna is a braided river with significantly different planform characteristics to the other major rivers passing through the country. General river characteristics of the other major rivers of Bangladesh are covered in the following sections.

7.2 THE GANGES RIVER

The Ganges river rises west of the Nanda Devi range west of Nepal and it is 2200 km upto Goalundo in Bangladesh. It has a drainage area of 977,500 sq.km.

The Ganges is a wide meandering river. Its width may be at places even 5 km. According to NEDECO (1967) the water level slope near the Harding Bridge is approximately $5x10^{-5}$. The Darcy-Weisbach resistance co-efficient of the Ganges varies between 0.08 for low flood condition to 0.01 for flood situation.

India has built a dam at Farakka on the river for diverting its winter flow. Pre-diversion average discharge in winter was 2265 m^3/s. And its average discharge in monsoon is 18,150 m^3/s. After the diversion maximum recorded discharge is 76,000 m^3/s (1987) and the minimum recorded discharge is 660 m^3/s (1976). Fig. 7.2-2 and Fig. 7.2-3 show some of the cross-sections and stages at the Ganges river for some selected years.

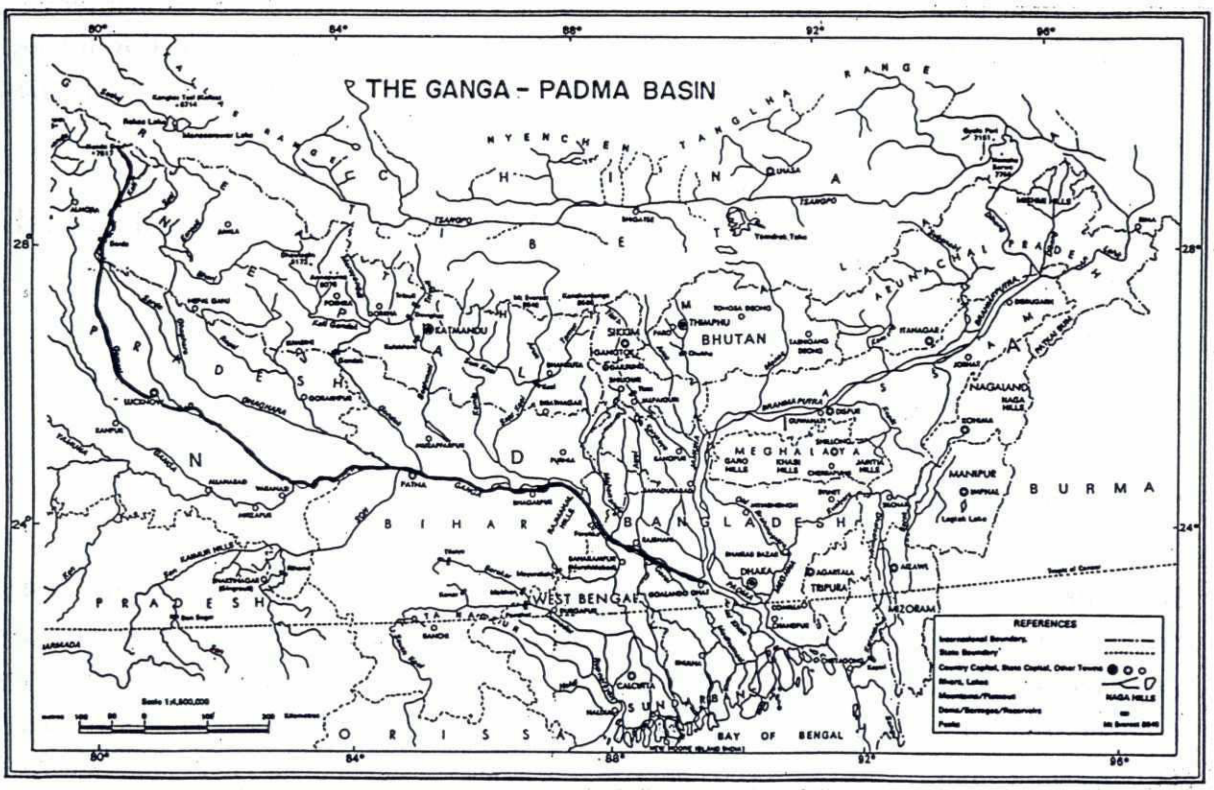

Fig. 7.2-1: The catchment area of the Ganges river

(Source: Verghese, 1990)

Total flow per year is estimated to be 3.62×10^{11} m^3. Its important distributories in Bangladesh are Bhagirathi, Boral, Gorai, Bhairab and Mathabhanga. Mohananda is one of its main tributaries. The average grain size (D_{50}) of bed material is 0.12 mm.

NEDECO (1983) estimated that suspended transport is the main mode of transport in the river. NEDECO (1983) processed the suspended sediment load data for the period 1963-1977. The result is shown in the following Fig. 7.2-4 along with a comparison of earlier published data of van der Veen (1962).

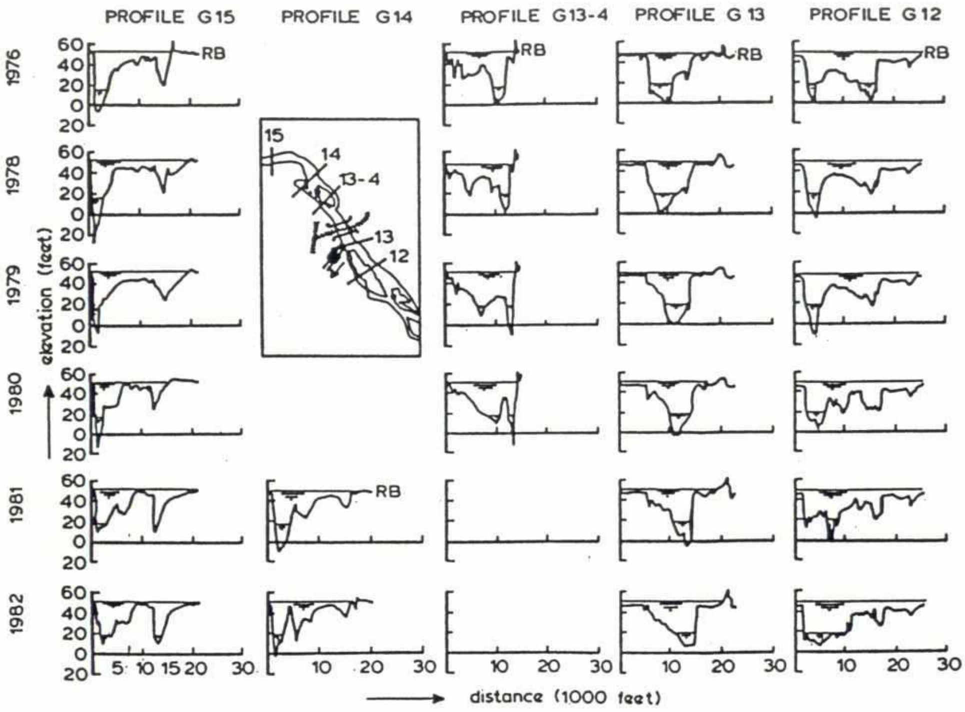

Fig. 7.2-2: Cross-sections of Ganges river near Hardinge Bridge

(Source: Baset Sarker, Klaassen, Radu, 1984)

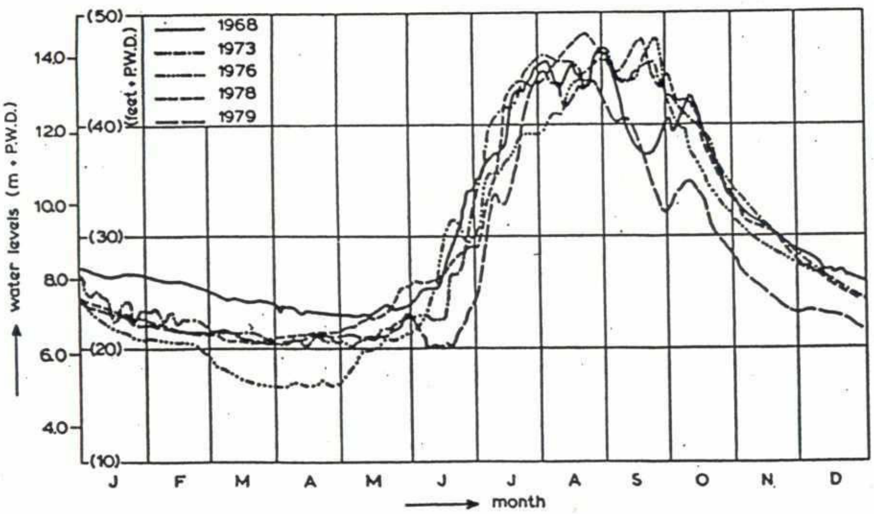

Fig. 7.2-3: Stages of Ganges river at Hardinge Bridge for some selected years

(Source: Baset Sarker, Klaassen, Radu, 1984)

NEDECO (1983) estimated a sediment load of 410 x 10^6 ton/year. It has been observed that morphological conditions of the river are changing continuously. This can be seen from the following Fig. 7.2-5 and Fig.7.2-6.

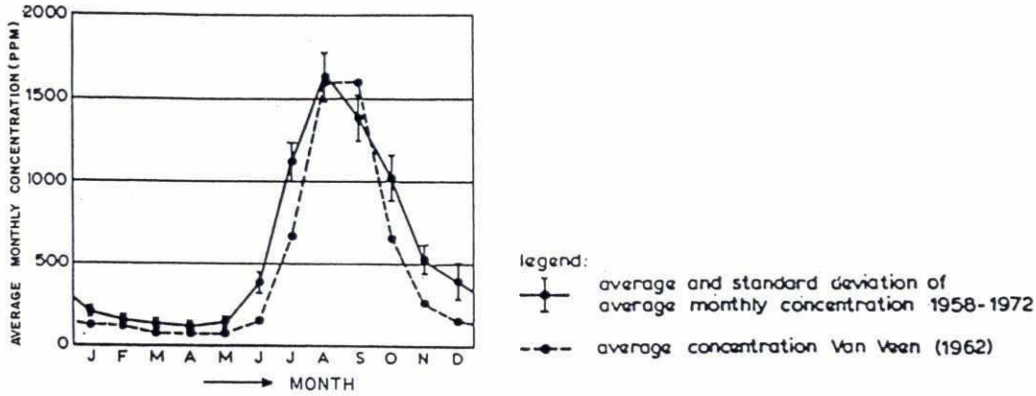

Fig.7.2-4: **Average and standard deviation of average monthly concentration of suspended sediment load in the Ganges river near Hardinge Bridge**
(Source: Baset Sarker, Klaassen, Radu, 1984)

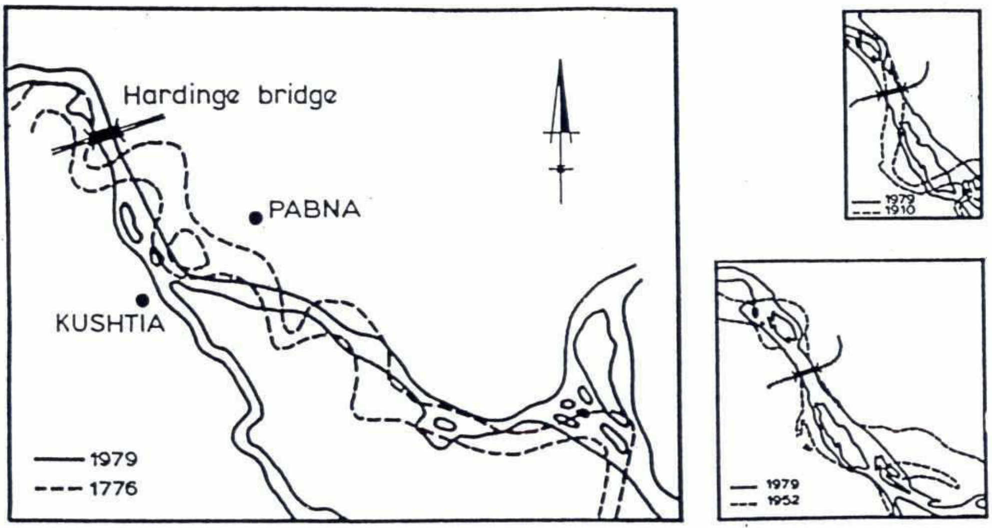

Fig. 7.2-5: **Changes in the course of the Ganges river between 1776 and 1979**
(Source: IECO, 1980)

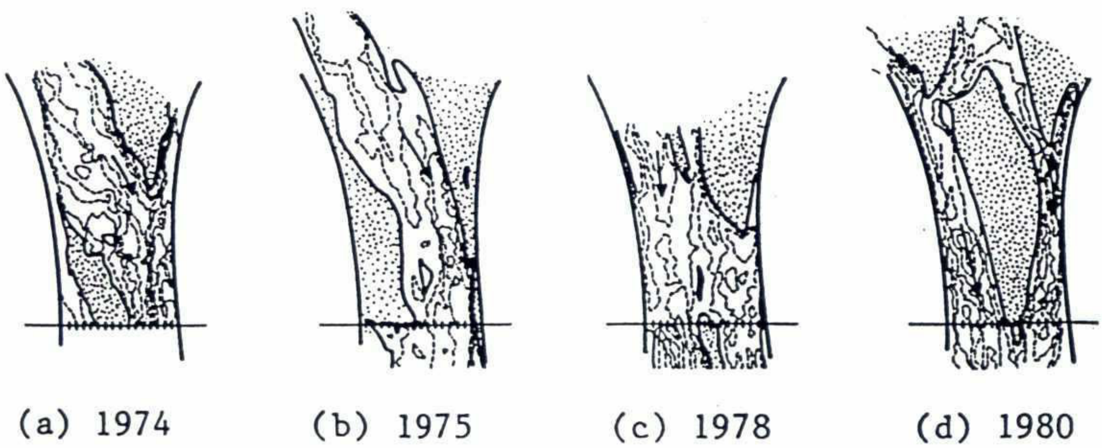

Fig. 7.2-6: **Pattern of channels and bars upstream of Hardinge Bridge according to soundings of Bangladesh Railways**
(Source: Baset Sarker, Klaassen, Radu, 1984)

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7.3 THE PADMA RIVER

The Padma river is the name of combined flow of the Ganges and the Jamuna. It is 120 km long and joins with the Meghna near Chandpur. Its width varies from 4-6 km. The Arial Khan river is one of the important distributories of the river. Fig. 7.3-1 shows the planform of the Padma river.

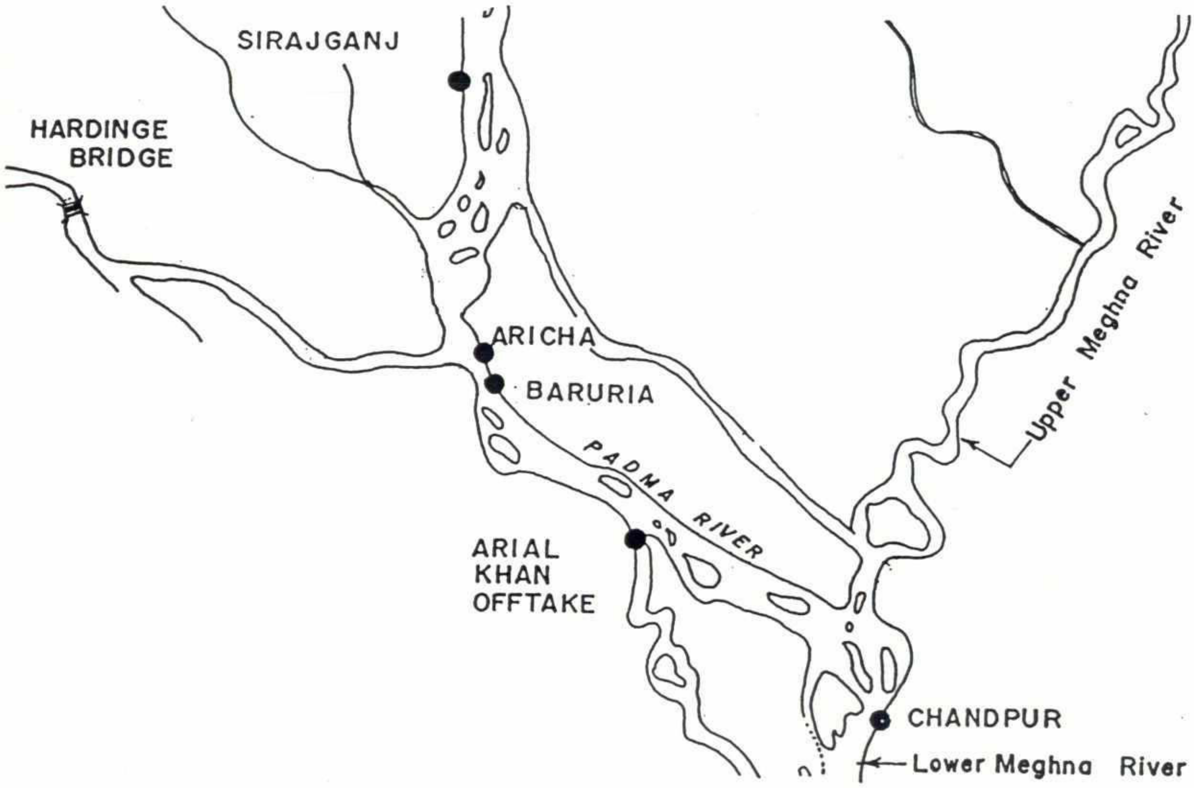

Fig. 7.3-1: Planform of the Padma River

(Source: RPT, NEDECO, BCL)

The Padma river is a stable braided channel with a channel slope of 0.2×10^{-4}. D_{50} of bed material is 0.13 mm. The flood plain is composed of clay of median size 0.03 - 0.045 mm. The difference of elevation between the flood plain and the channel bed is $\pm$ 10 m.

7.4 UPPER MEGHNA RIVER

The Upper Meghna river flows in the northeastern part of Bangladesh as can be seen in Fig. 7.4-1. The Upper Meghna carries the combined flow of the Surma and the Kusiyara rivers which originate in the Indian hills northeast of Bangladesh. The Surma river flows through the Sylhet area which is rapidly sinking away. A number of tributaries of the Surma originate from the Silong hills in India and from piedmont areas. They bring quite some sediment (boulders, gravel, sand), that mostly settles in the Sylhet area.

Going in downstream direction, the Meghna river is joined by the Old Brahmaputra at Bhairab Bazar. The Dhaleswari is another tributary and it joins the Meghna river at

Munshiganj on the right bank. Some tributaries that originate from the Tripura hills join the Upper Meghna at the left bank. Although having some reaches with a system of various channels, the Upper Meghna river can be characterized as a river mainly meandering within a well defined high water bed and having flood discharges up to some 20,000 m^3/s and annual discharge is $1.13x10^{11}$ m^3.

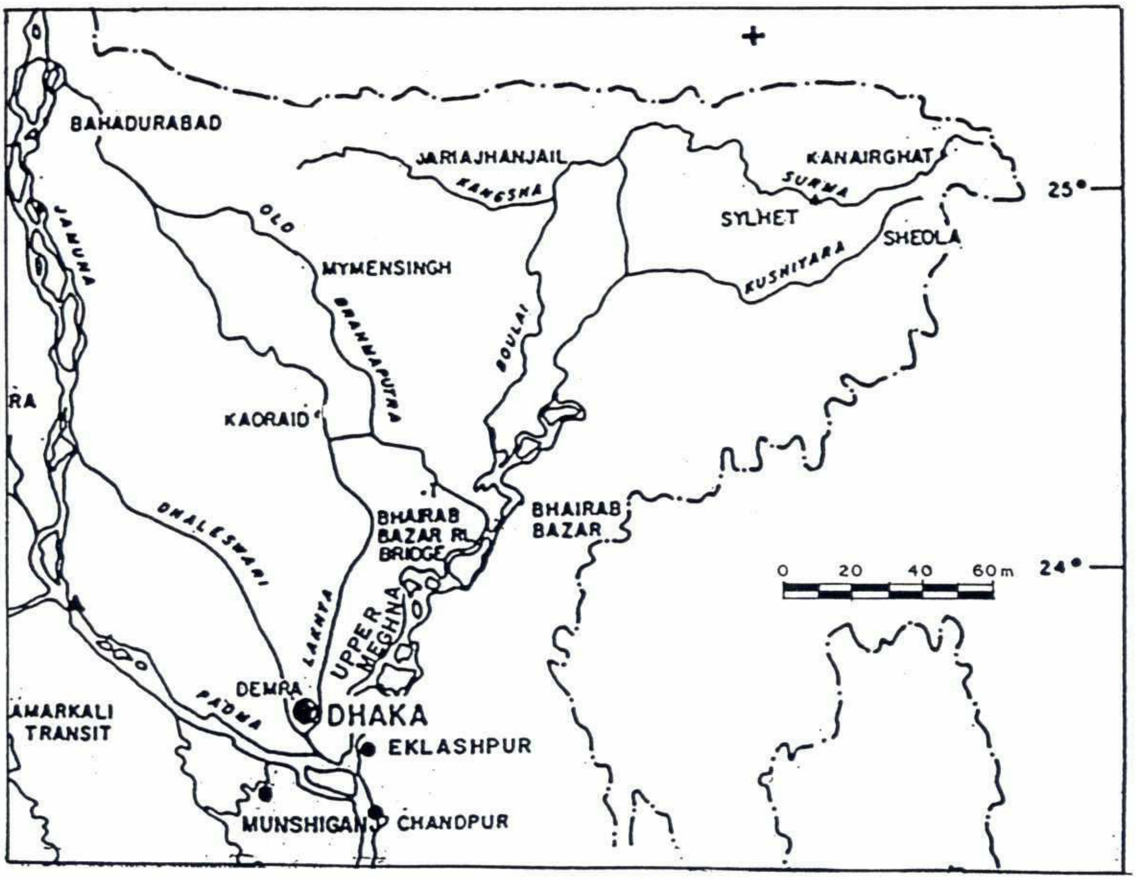

Fig. 7.4-1: The Upper Meghna river

(Source: Haskoning, Delft Hydraulics, BETS, 1992)

The Upper Meghna flows through probably, structurally, the most active part of the Bengal basin, notably the Sylhet depression, that has subsided about 10 meters within the last several hundred years. This subsidence may be a combination of compaction together with tectonic activity (related to the major fault system bounding the northern side of the area). Together with the compaction, the tectonic activity is another factor affecting the drainage of the Sylhet area. This deeply flooded area is in fact an area which is inundated by the backwaters of the Ganges and the Jamuna. Because of the high water levels in these rivers, the floods from the Sylhet area do not have sufficient slope to be rapidly evacuated to the Bay of Bengal. This "impeded drainage" causes the floodwaters to accumulate in the deeply flooded area, causing silting up, and so creating an enormous basin which only gradually drains into the Meghna. The prevailing morphological action in this upper catchment area is sedimentation rather than erosion (Hoar Development Reconnaissance Study, 1986).

The river is about 110 km long and three quarter to one kilometer wide. The average daily water levels vary between 0.81 m and 7.65 m. Tidal influence is about 1 m at its confluence with the Padma. The duration curve of the daily discharge is given in the following Fig.7.4-2.

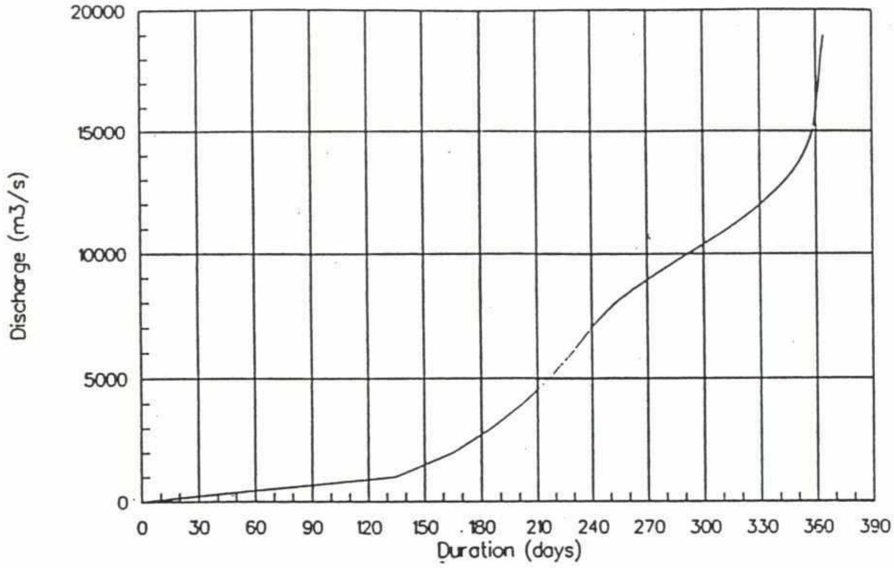

Fig. 7.4-2: Duration curves of the daily discharges at Bhairab Bazar

(Source: Haskoning, Delft Hydraulics, BETS, 1992)

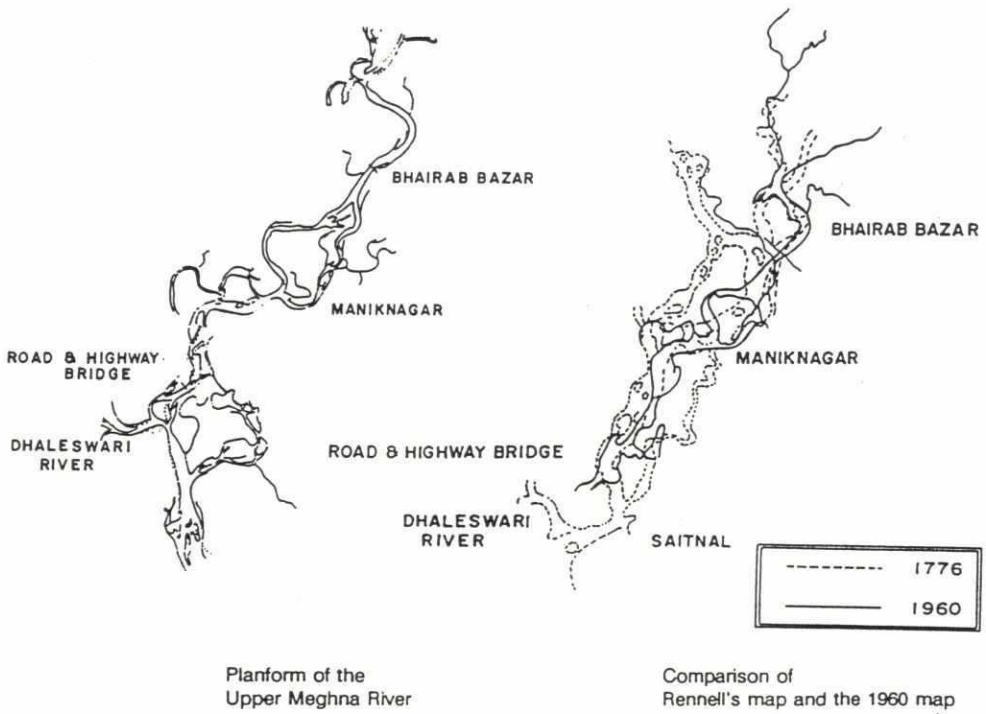

Fig. 7.4-3: Adaptation of the bed of the Upper Meghna river

(Source: Haskoning, Delft Hydraulics, BETS, 1992)

The planform of the river consists mainly of one single channel and over two reaches two parallel ones of similar importance. There are many abandoned channels or channels which carry no water during the low flow season. The reason for this may be that the river is flowing in a bed which has been shaped by the flow of the Brahmaputra river before it changed its main flow to the Jamuna. The present river is still in the process of adapting its bed to the new ultimate condition. This can be seen from the Fig. 7.4-3.

It can be assumed from an observation of the figure that in and around 1776 the behaviour of the Upper Meghna had a braided pattern like the present Jamuna.

Water level slope of the Upper Meghna river is very small 0.2×10^{-4}. The bankfull discharge is about 8000 m^3/sec. and dominant discharge of about 11,000 m^3/sec. The D_{50} of the river is 0.14 mm.

7.5 LOWER MEGHNA RIVER

The Lower Meghna carries the combined flow of the Padma and the Upper Meghna rivers. The Padma river in turn carries the discharge of the Brahmaputra and the Ganges river. Hence the Lower Meghna river is one of the largest rivers in the world. The river flows from the confluence of the Upper Meghna river and the Padma river via Chandpur into the Bay of Bengal (see Fig. 7.5-1).

The Lower Meghna river system is characterized by a wide river bed of several kilometres in which various channels develop in combination with large propagating sand bars (locally called chars). A very peculiar aspect is the sudden turn to the south of the Lower Meghna river just downstream of the confluence with the Upper Meghna river. This sharp turn has been present for a very long time and may be the cause of the gradual shift of the Ganges river, and later the combined Ganges and Jamuna system, in easterly direction. The overall tendency of the Lower Meghna river is erosion on the eastern bank and accretion on the western bank.

Smaller tributaries join the river on the left side downstream of the confluence with the Upper Meghna river, notably the Gumti river and the Dhakatia river.

Along the left bank of the Lower Meghna at some sites erosion causes severe problems. Going in downstream direction, problem sites are or have been in the recent past: Eklashpur, Chandpur and Haimchar.

The average daily water level of the river varies between 0.56 m to 4.99 m at Chandpur. The river is subjected to tidal influence. During low flow vertical tide of some 1.6 m is noticeable in Chandpur.

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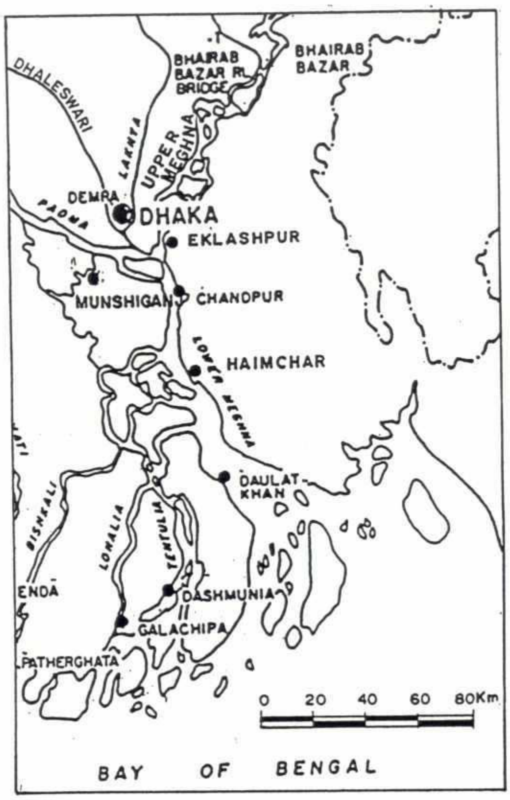

Fig. 7.5-1: Planform of the Lower Meghna river

(Source: Haskoning, Delft Hydraulics, BETS, 1992)

The average slope of the river is around $0.5x10^{-4}$. The discharge varies between 10,000 m^3/sec to 160,000 m^3/sec. The dominant discharge is about 80,000 m^3/sec which is slightly lower than the bankfull discharge of 82,500 m^3/sec. A tentative duration curve for the daily discharges at Chandpur is given in the following Fig. 7.5-2. The D_{50} of bed material is 0.09 mm.

The large bend downstream of the confluence has a radius of curvature of some 15 km. At the inner side of this bend a large point bar appears to be present. Here the river narrows again to a few kilometres only. The geometry of the confluence has changed substantially over the last three decades, mostly due to the continuing bank erosion along the outer bend. As can be observed from maps and satellite images the islands that are generated downstream of Mawa vanish when entering the bend. During some years a "southern" channel is present in the bend, that acts as a short cut.

Chandpur is located at the lower end of this bend. About 15 km downstream of Chandpur Haimchar is located. Near Haimchar a large bend used to be present but over the last two decades a natural cutoff has occurred, and now the river reach near Haimchar is essentially straight.

From an analysis Klaassen (1990) observed that bed material of the river is always in movement. He also observed that dominant mode of transport is apparently the suspended load. He further calculated the yearly bed material transport to be around 70×10^6 ton.

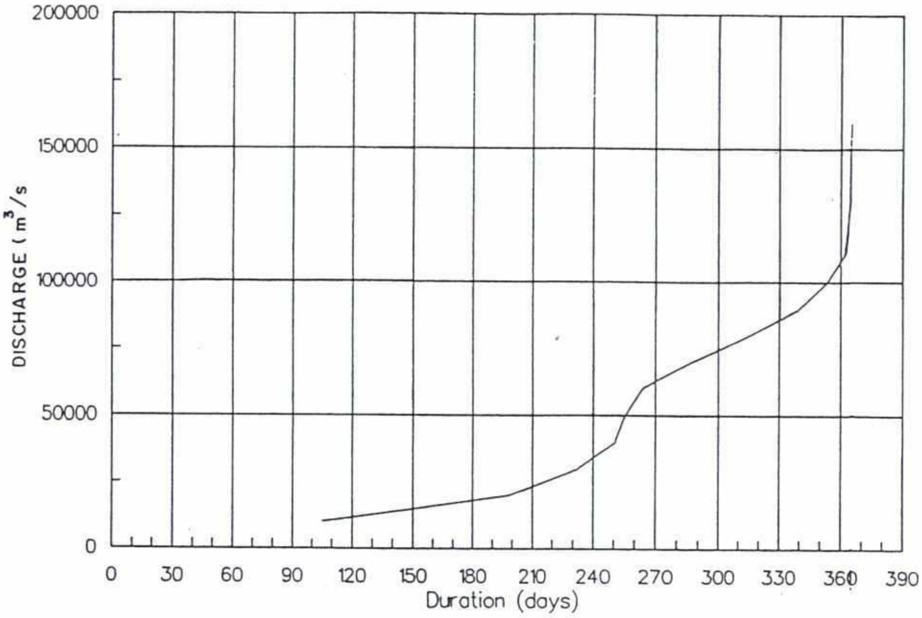

Fig. 7.5-2: Tentative duration curves of the daily discharges at Chandpur

(Source: Haskoning, Delft Hydraulics, BETS, 1992)

7.6 SUMMARIZED COMPARISON

Table 7.6-1 provides a comparison of some of the river characteristic for major rivers in Bangladesh :

Name of River	Catchment Area (sq.km.)	Annual Average Discharge (m^3/s)	Slope	D_{50} of Bed Material (mm)
Jamuna	560,000	19,462	0.7×10^{-5}	0.25-0.16
Ganges	977,500	10,874	0.5×10^{-5}	0.12
Padma	1,560,000	-	0.2×10^{-4}	0.13
Upper Meghna	77,000	4,800	0.2×10^{-4}	0.14
Lower Meghna	1,637,000	-	0.5×10^{-4}	0.09

Table 7.6-1: Comparison of the characteristic for major rivers in Bangladesh

Note: D_{50} - mean grain size

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