FLOOD PLAN COORDINATION ORGANIZATION (FPCO)

KREDITANSTALT FÜR WIEDERAUFBAU (KfW)

CAISSE CENTRALE DE COOPERATION ECONOMIQUE (CCCE)

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

INTERIM REPORT

Volume II : Annex 1
(Technical Report No.2)

River Training and Morphological Response

JULY 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)
DESH UPODESH LIMITED (DUL)

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

ANNEX 1

(TECHNICAL REPORT NO.2)

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

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PRELIMINARY SELECTION OF RECURRENT MEASURES

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FOREWORD

(i) REFERENCE

This report on the river training and morphological response has been prepared as Technical Report No.2; of the Bank Protection and River Training (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
- 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 Consulting Agreement) are :

- o to formulate active flood plain management (AFPM) measures and strategies narrowing the scope of river training in this report, (see Chapter 1)
- 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 plays 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 (see for instance Inception Report Fig. 4.4.3-4)

- **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 ... 30 as per Consulting Agreement this Technical Report No.2 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.

Summarizing it can be stated that the report is an elaboration of the sections 4.4.2 and 4.4.3 of the Inception Report emphasizing the hydraulic and morphological effects of recurrent measures.

PART A

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

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

1. INTRODUCTION

1.1 OBJECTIVE

The "State-of-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. 2-1).

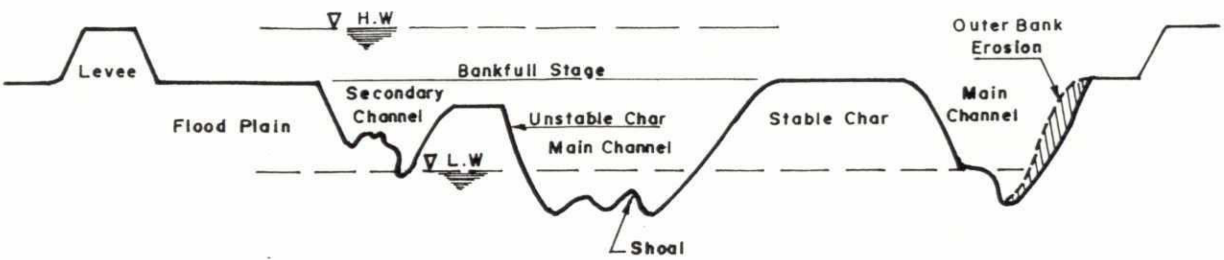

Fig. 2-1 : Typical cross-section of a braided river

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

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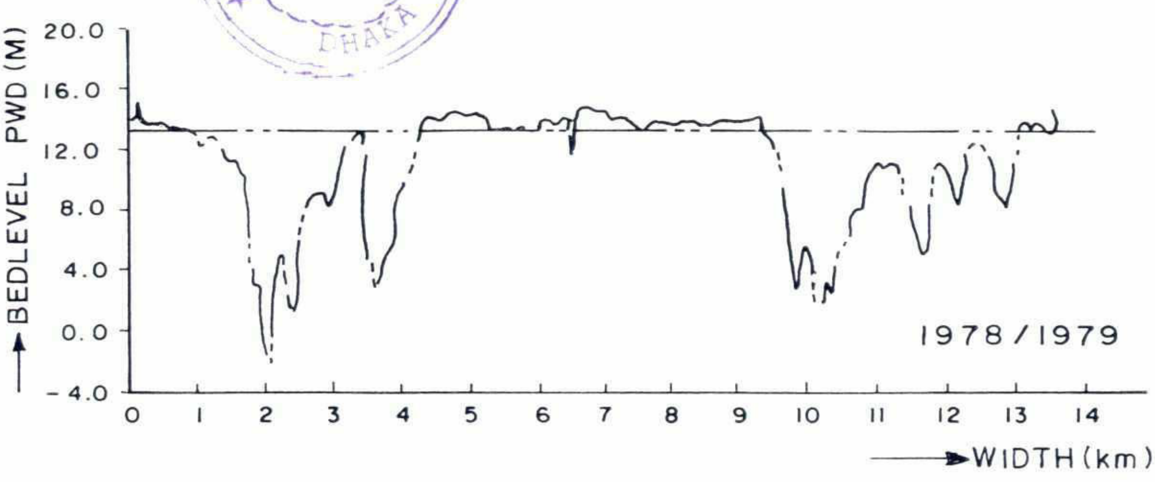

Fig. 2-2 : Typical cross-section Jamuna River (RPT 1989)

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 disappear, (see sketch above and section hereafter)

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 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.

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

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

- o **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)

- o **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 upto 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.

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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).

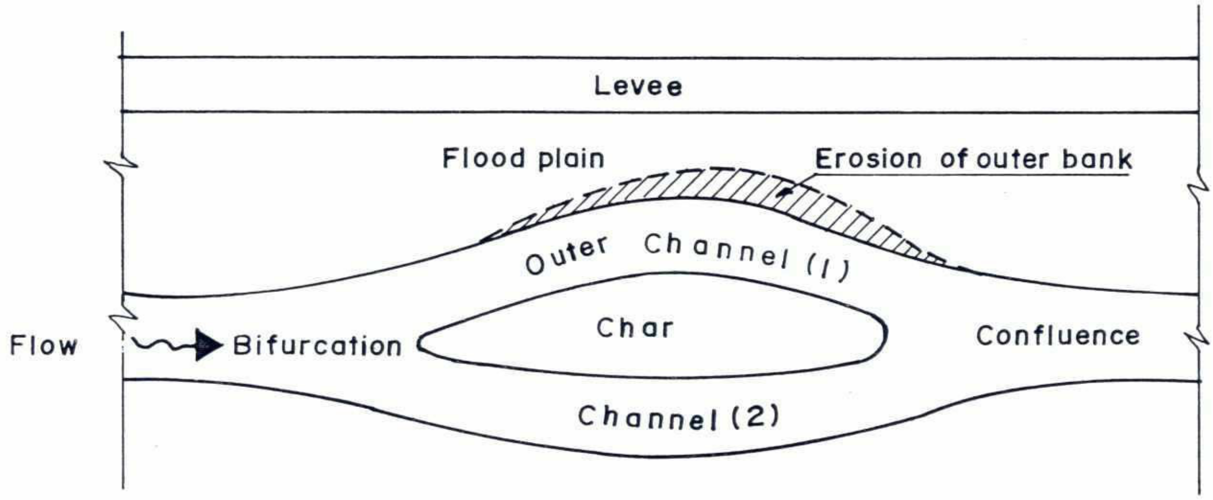

Fig. 2-3 : Typical planform

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.

2.4.1 Flow redistribution at bifurcation

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:

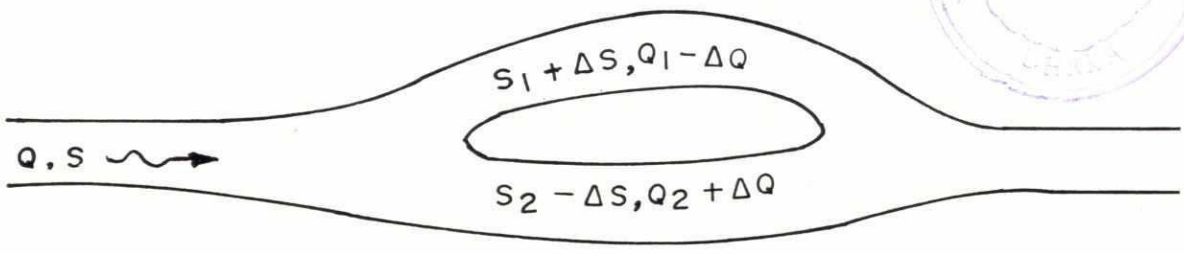

Fig. 2-4 : Definition sketch

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

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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.

- **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 required study, (see further section 2.5).

2.4.2 Flow Redistribution in Cross-section

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 sketches hereafter).

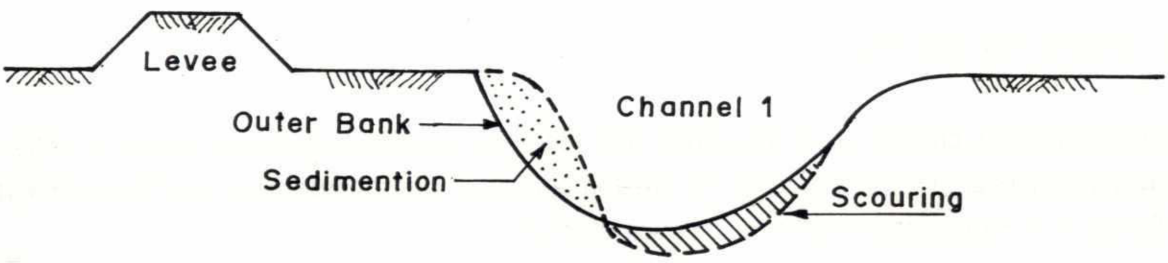

Fig. 2-5 : Sedimentation at the outer bank

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. 2-6.

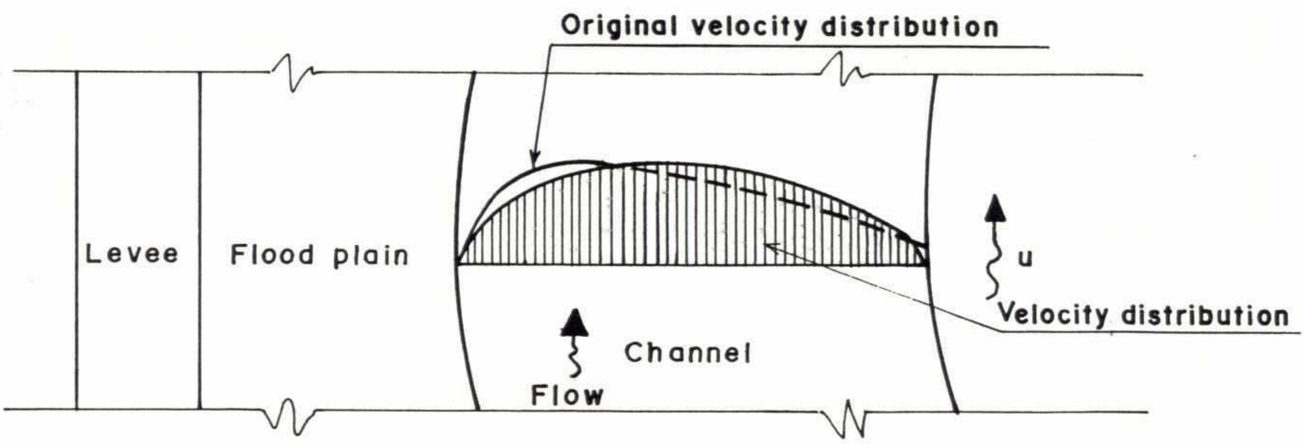

Fig. 2-6 : Changing flow distribution

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 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; 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

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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.

3.1 PERMANENT OR RECURRENT

As stated before the terminology "permanent and recurrent" indicate the possible lifetime of the measures. Permanent means long term upto say 50 years (engineering time scale), recurrent means short term upto 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 3-1 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 3-1: 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. 3-1). 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.

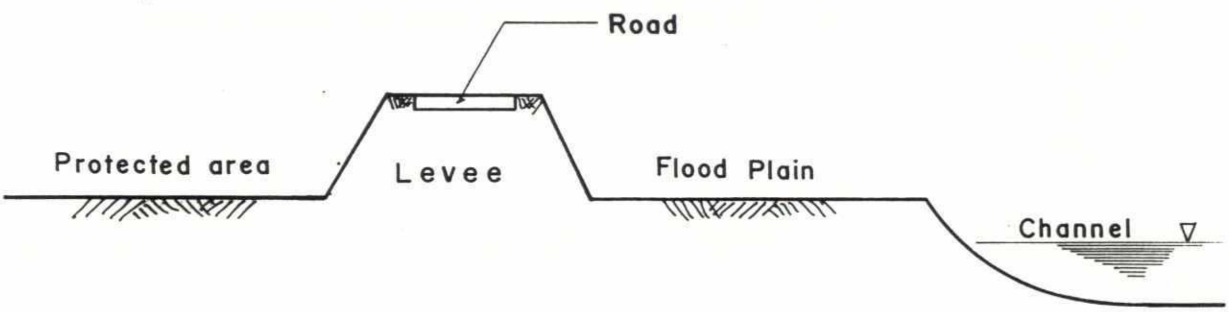

Fig. 3-1: Location of levee

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).

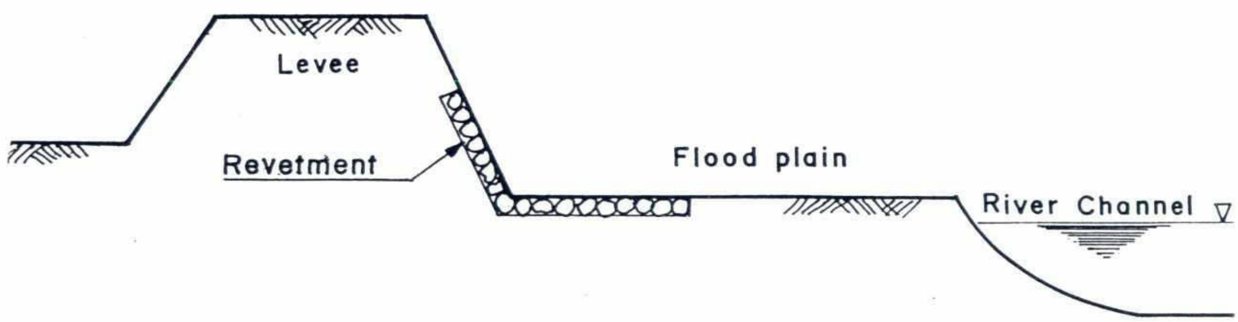

Fig. 3-2: Revetment on flood plain

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.

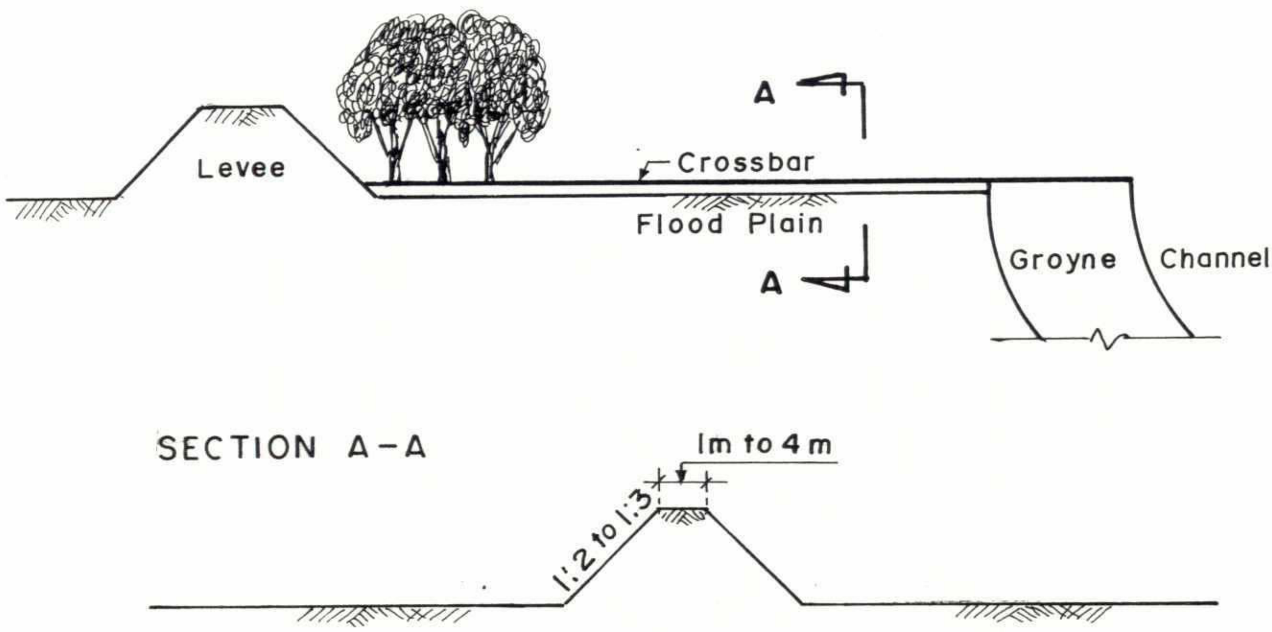

Fig. 3-3 : Crossbars

Fig. 3-3 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 Capital Dredging

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 (see Fig. 3-4)
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 upto 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.

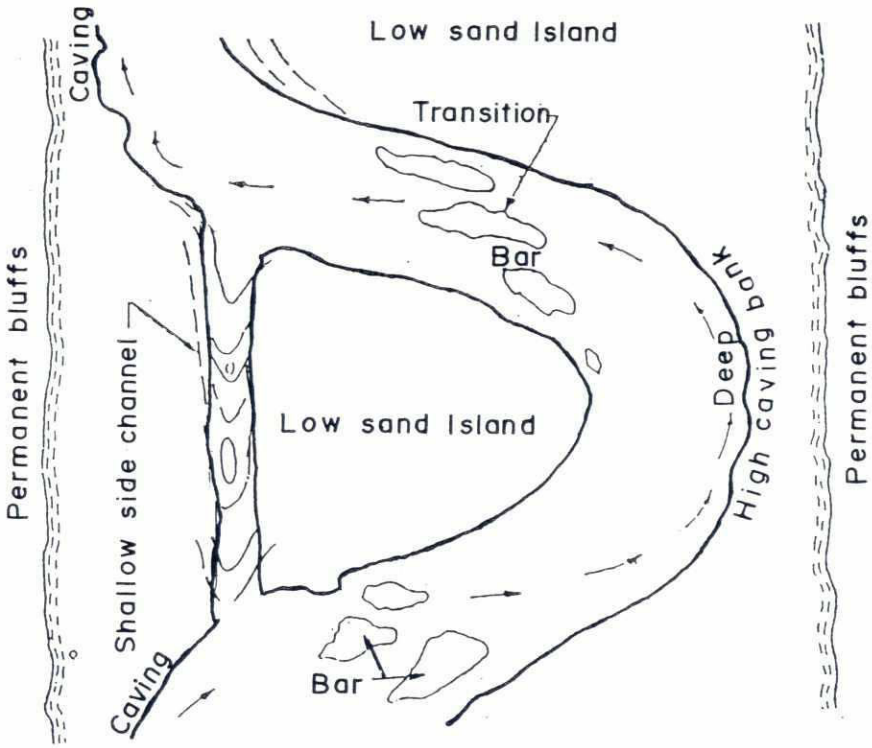

Fig. 3-4: Natural Cut-off
(Source: Joglekar, 1971)

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. 3-5).

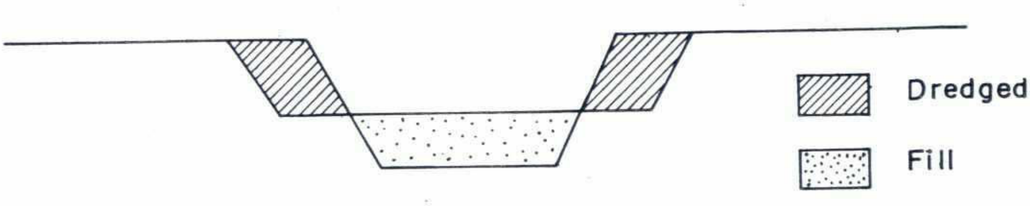

Fig. 3-5: River confinement

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

For these type 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 (source IHC Holland) have been summarized in Table 3-2.

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: Characteristics of suction dredgers

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

30

An example of the possible output of a Beaver 1200 for 10 m dredging depth the Fig. 3-6 is presented.

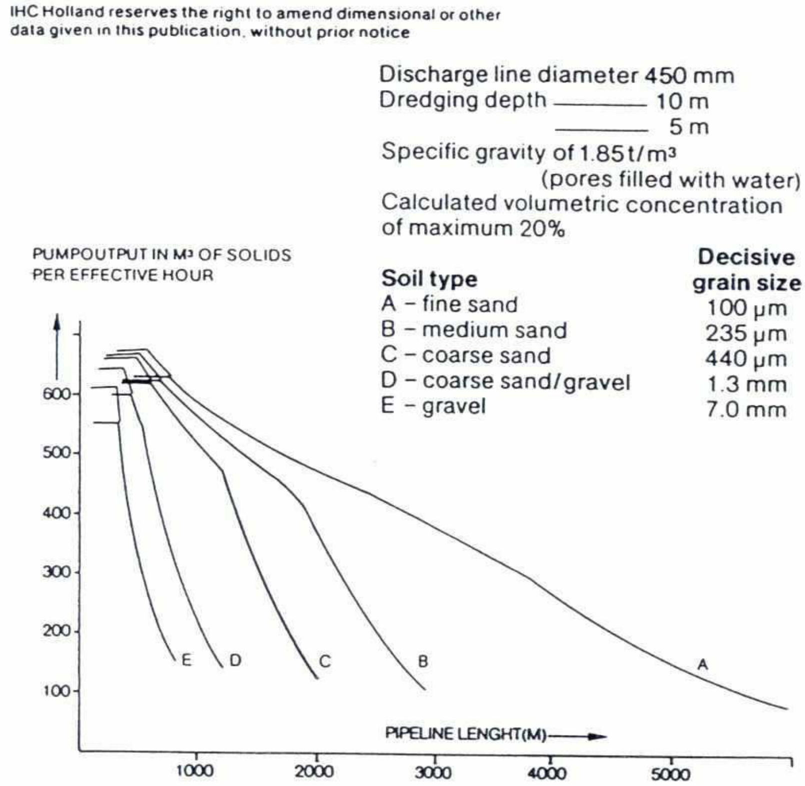

Fig. 3-6: Output of a Beaver 1200
(Source: IHC, 1988)

In Bangladesh the maintenance of waterways is undertaken by BIWTA using 8 dredgers with the output as presented in Table 3-3 (Source : DHV, 1988).

Type of Dredgers	No of Dredgers	Output in m³ Solids per Dredgers			Totals (m³/year)
		Mean Average Day Production	Monthly	Annual	
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 3-3: 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. 3-7).

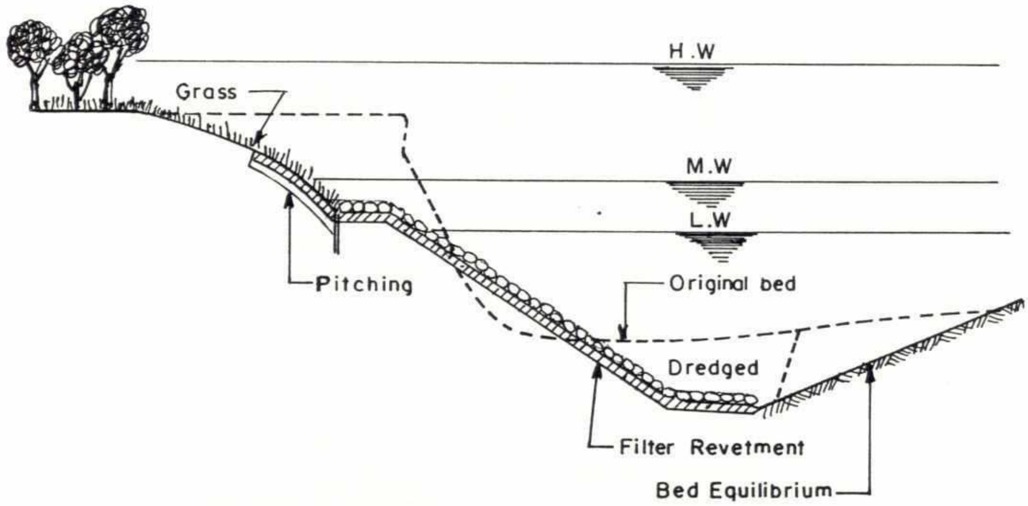

Fig. 3-7: Bank revetment
(Source: Jansen et al., 1979)

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. 3-8)

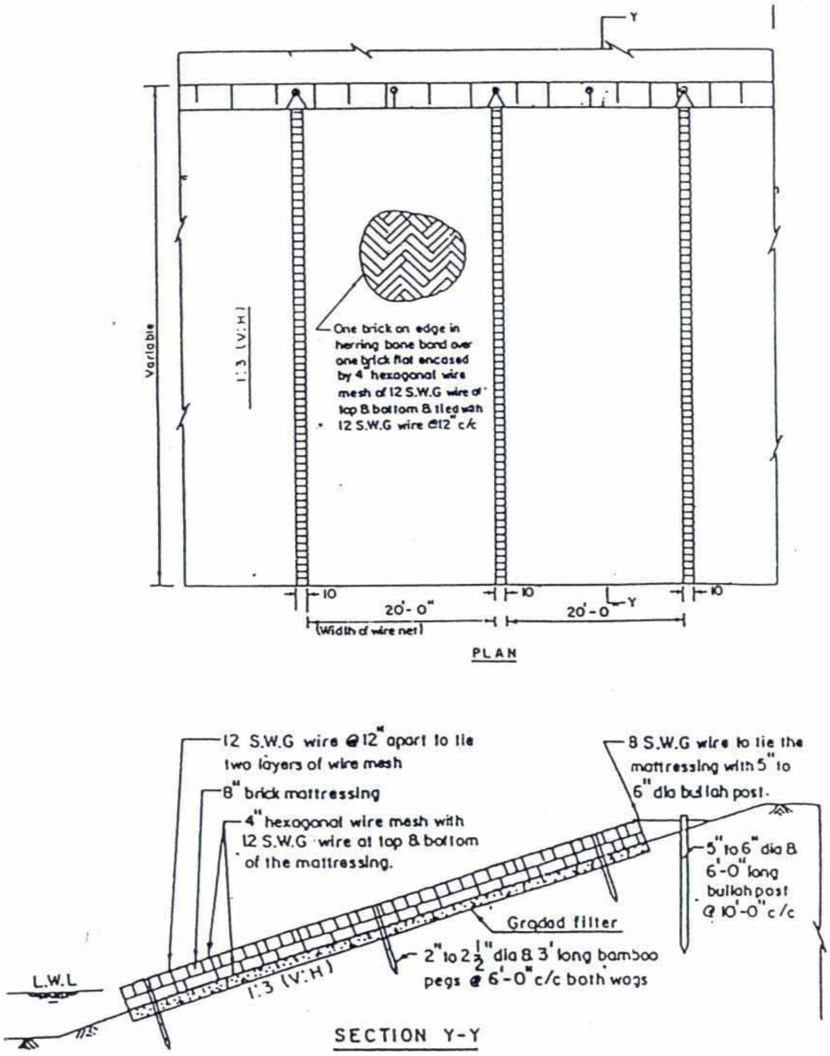

Fig. 3-8: Details of brick mattressing
(Source: Huq, 1988)

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.

Articulated concrete slabs over a filter layer have been found effective.

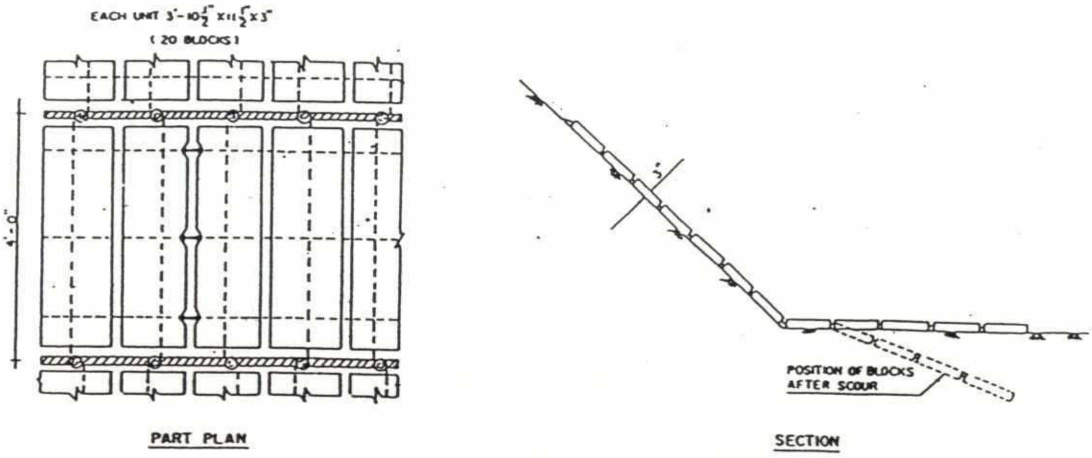

Fig. 3-9: Details of articulated concrete slab
(Source: Huq, 1988)

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. 3-10), 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 upto 10 m. Groyne length 30 m. Groyne Spacing 100 m.

Concrete working bridge on top of screen cum stability beam.

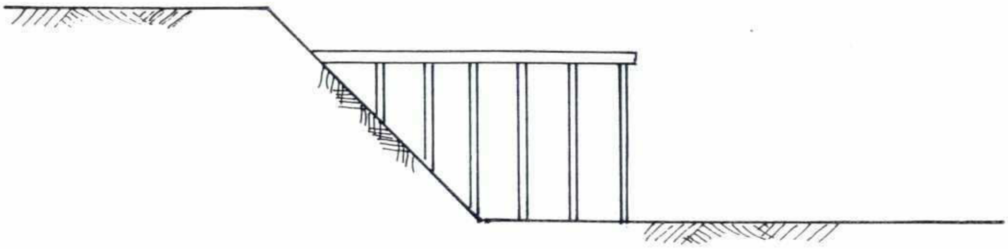

Fig. 3-10: Needle groyne

(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. 3-11). 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. 3-12).

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

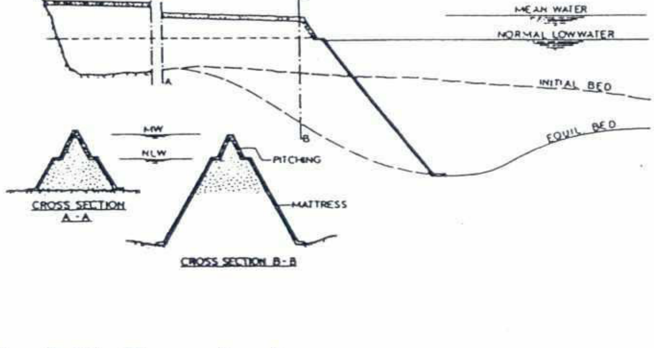

Fig. 3-11: Example of a groyne
(Source: Jansen et al., 1979)

Fig. 3-12: Flow pattern between groynes
(Source: Jansen et al., 1979)

These type of groynes are permanent in nature and involve huge cost for construction. Sometimes river length can be stabilised by constructing series of groynes at appropriate places on both banks (see Fig. 3-13).

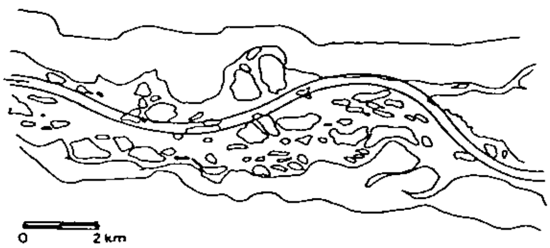

Fig. 3-13: Channel regulation on the Rhine downstream of Basle
(Source: Jansen et al., 1979)

Sometimes regulation is realised by a combination of groynes and dredging work (for the channel cut-offs, see e.g. Fig. 3-14 and 3-15) :

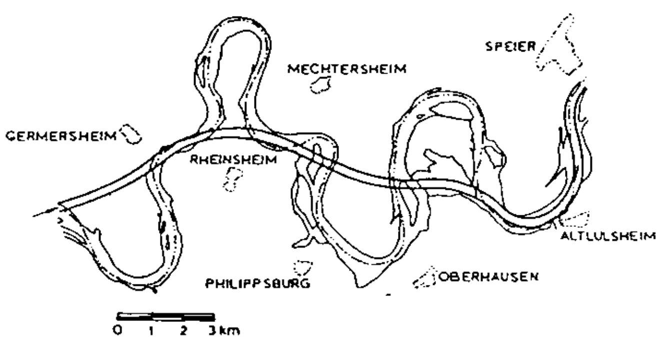

Fig. 3-14: Channel regulation on the Rhine upstream of Mannheim (19th Century)
(Source: Jansen et al., 1979)

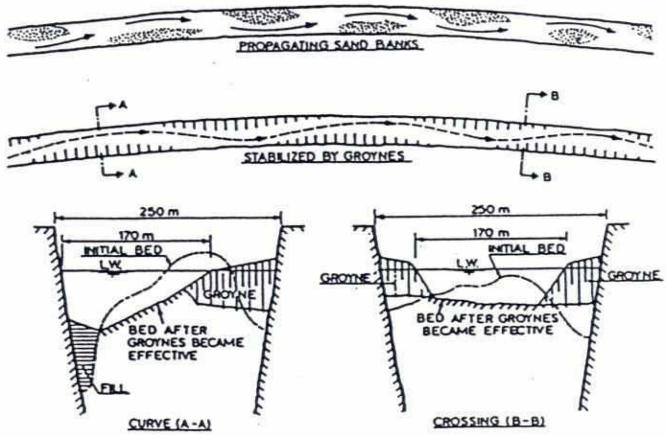

Fig. 3-15: Improvement of Rhine rectification by groynes (Source: Jansen et al., 1979)

(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. 3-16.

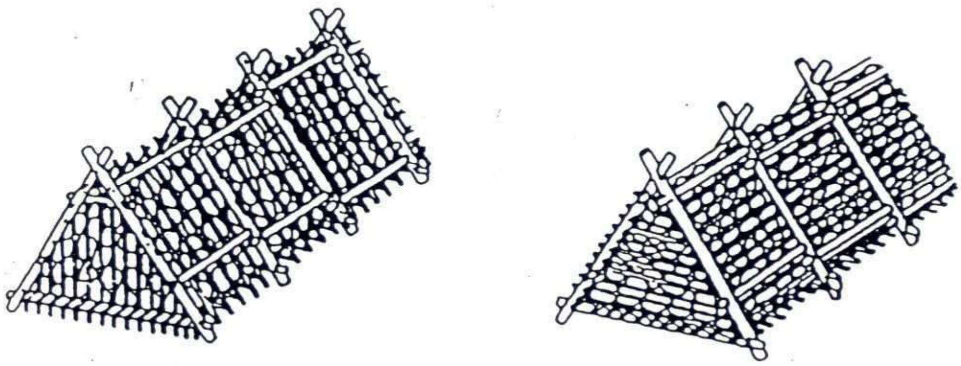

Fig. 3-16: Triangular Prism Crib (Source: UN, ECAFE, 1953)

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. 3-17).

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. 3-18).

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.

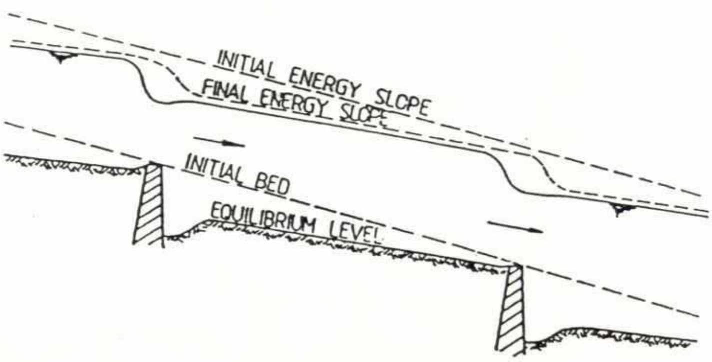

Fig. 3-17: Bed fixation by sills
(Source: Jansen et al., 1979)

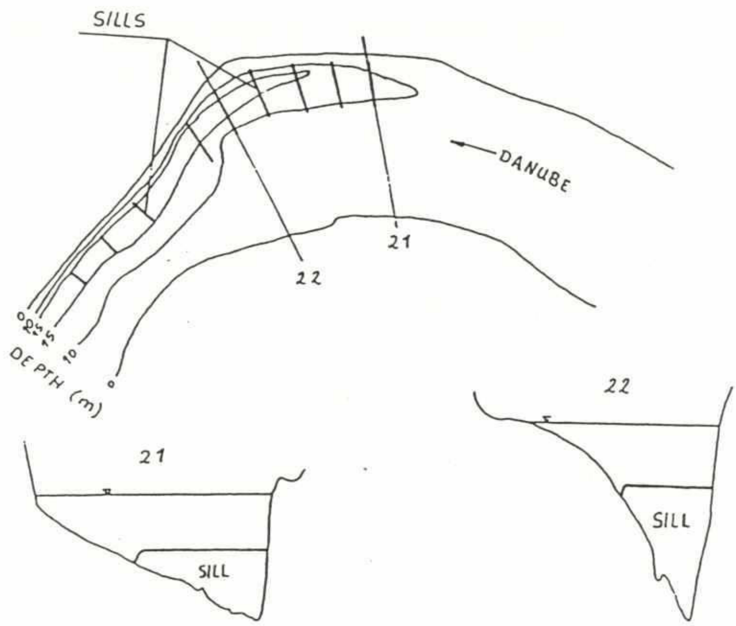

Fig. 3-18: Location of submerged sills on sharp bend of the Danube River
(Source: Pilarczyk et al., 1990)

(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. 3-19).

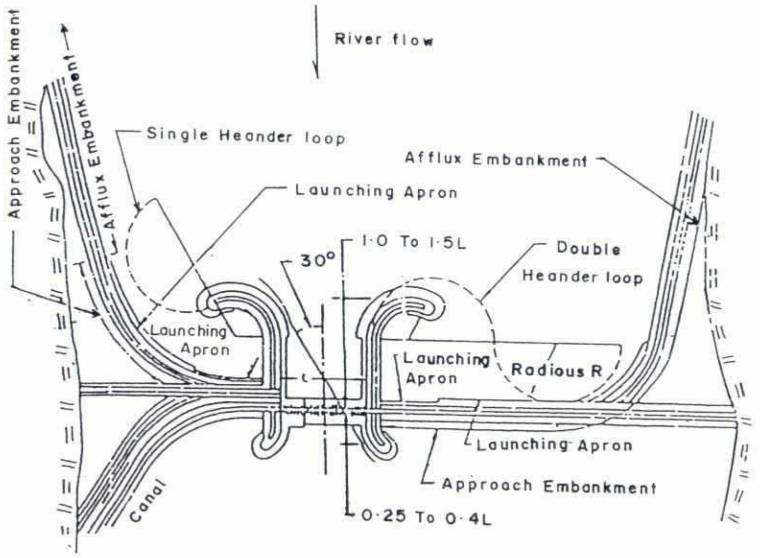

Fig. 3-19: Plan of a typical guide bund
(Source: IS: 8408, 1976)

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 redirect the overflow more perpendicular to the crest. The orientation of the vane is therefore fundamentally different from the crib.

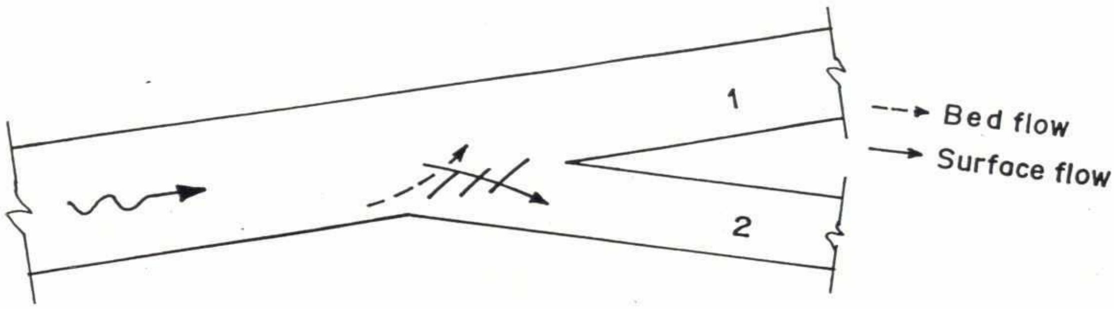

Fig. 3-20: Bottom vanes at a channel entrance

Fig. 3-20 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. 3-21) is presented.

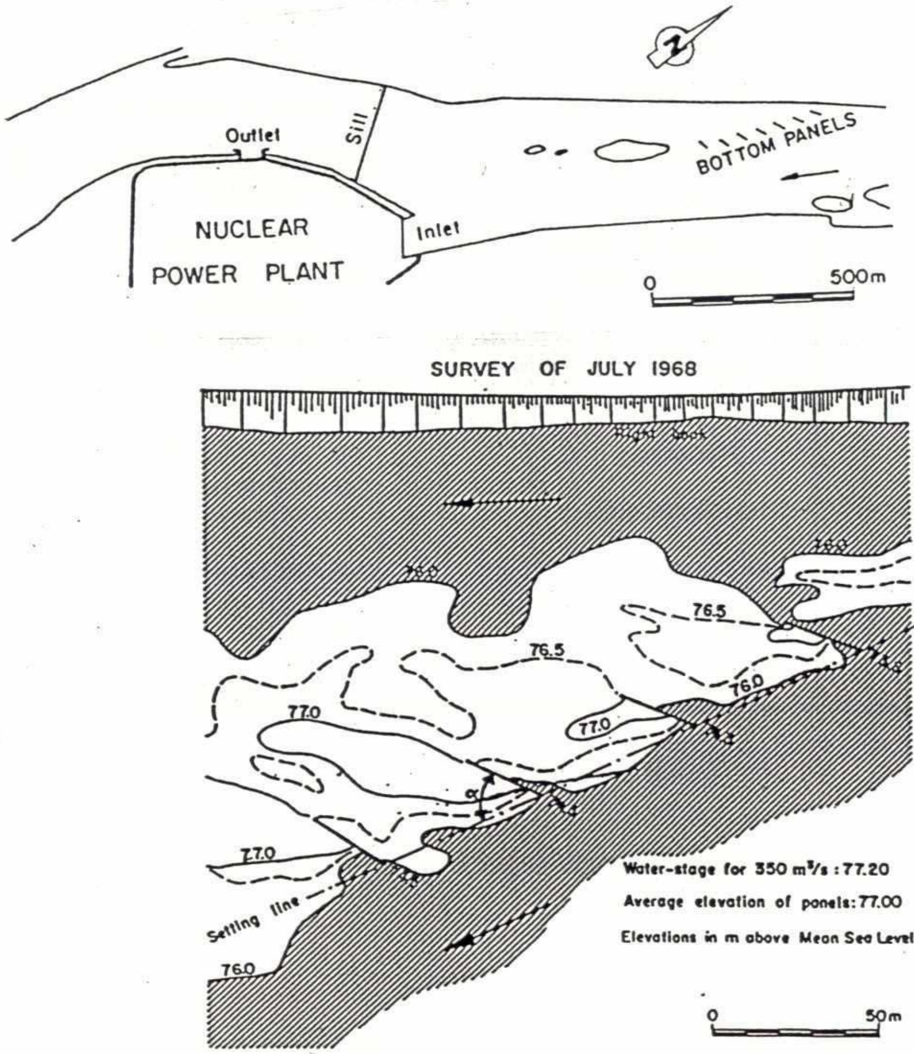

Fig. 3-21: Bottom level elevations due to bottom vanes 6 months after construction (Source: Remillieux, 1972)

The purpose of the bottom vanes (see Fig. 3-21) 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 3-4 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 - intermediate vanes	 such as the Chao Phraya screens 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 3-4: 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. 3-22)

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

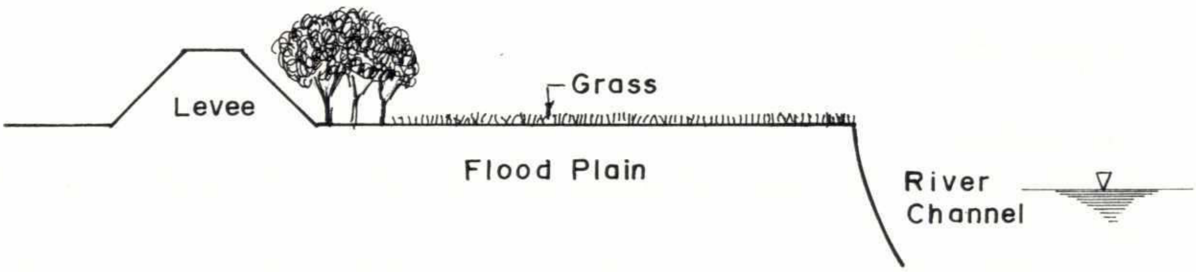

Fig. 3-22: Vegetation on the flood plain

The resistance of grass mats on the flood plain decreases as a function of time, see Fig. 3-23:

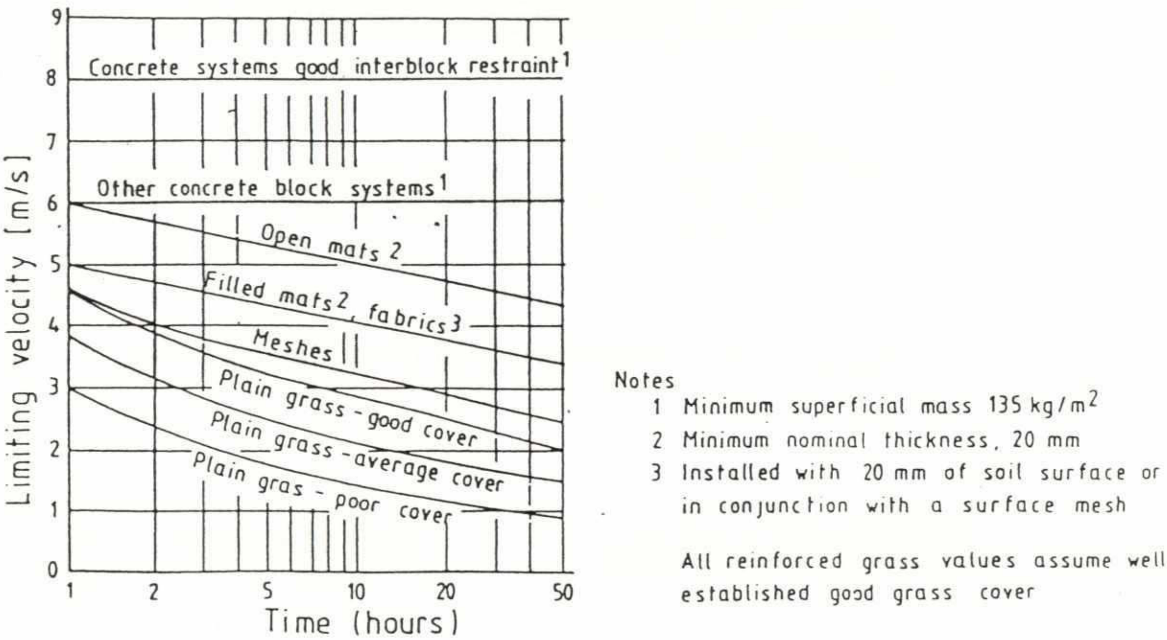

Fig. 3-22: Recommended limiting velocities for erosion resistance of plain and reinforced grass against unidirectional flow (after Hewlett et al. 1987) (Source: Pilarczyk et al., 1990)

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 fasciae rolls which are brush sausages filled with e.g. stones (see Fig. 3-24)
- o bush fence (see Fig. 3-25)

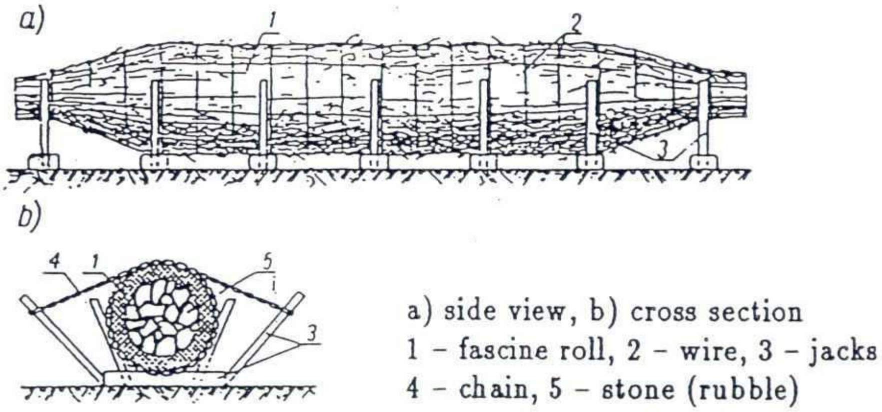

Fig. 3-24: Stone-filled fasciae roll (after Mamak, 1964)

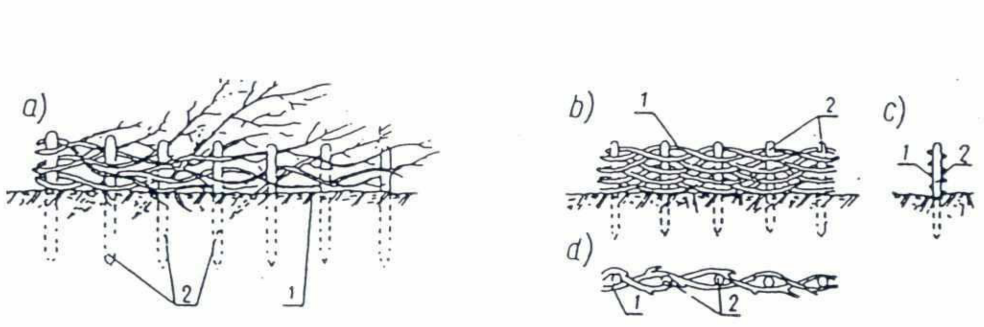

Fig. 3-25: Willow fence (after Mamak, 1964), a) during construction, b,c,d) after completion, 1 - willow, 2 - stakes

(Source for Fig. 3-24 & 3-25: Pilarczyk et al., 1990)

3.3.2 Maintenance Dredging

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

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

- bucket dredger
- cutter (suction) dredger
- dustpan (suction) dredger
- hopper (suction) dredger
- 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.

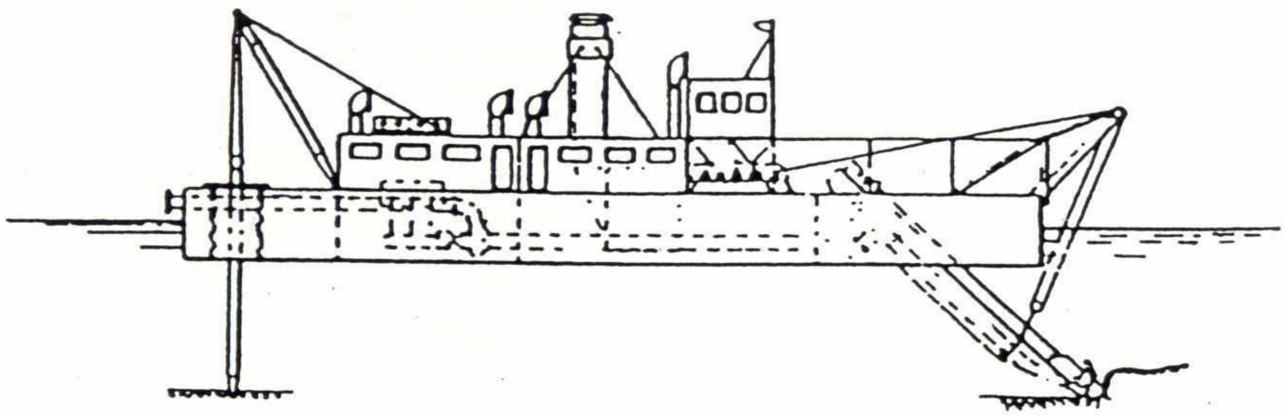

Fig. 3-26: Cutter suction dredger
(Source: Jansen et al., 1979)

So the cutter suction dredger (see Fig. 3-26) 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 (see Fig. 3-27 ... 3-30):

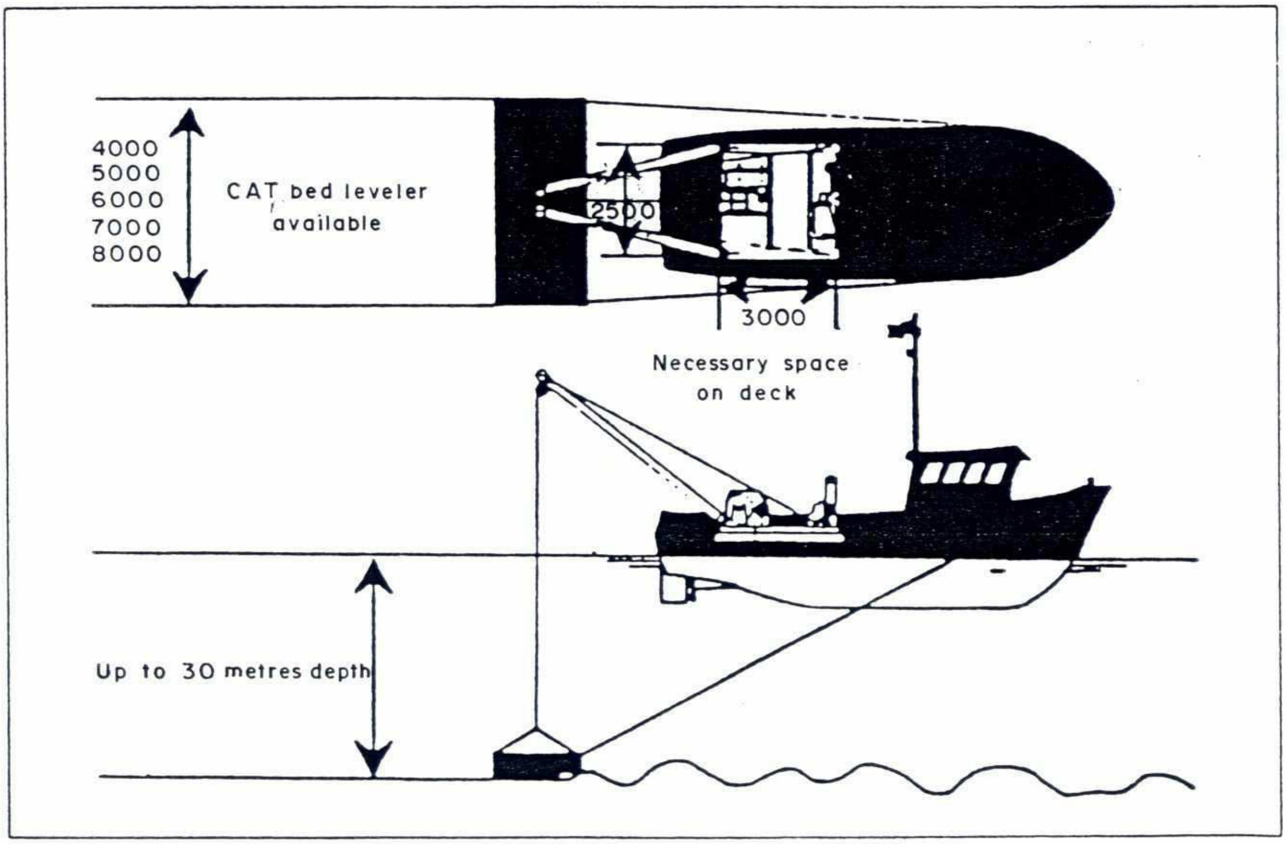

Fig. 3-27: Leveller towed behind a tug
(Source: UN, ESCAP, 1989)

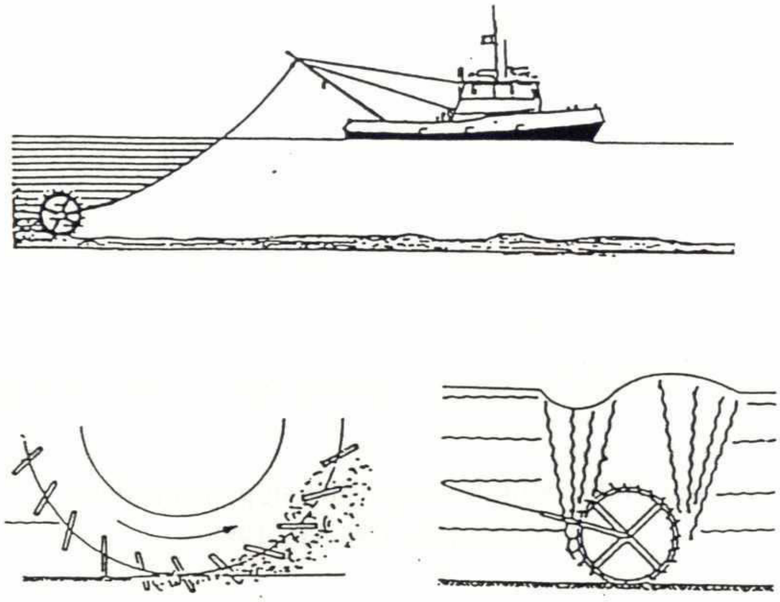

Fig. 3-28: Principle of operations of mudwheels
(Source: UN, ESCAP, 1989)

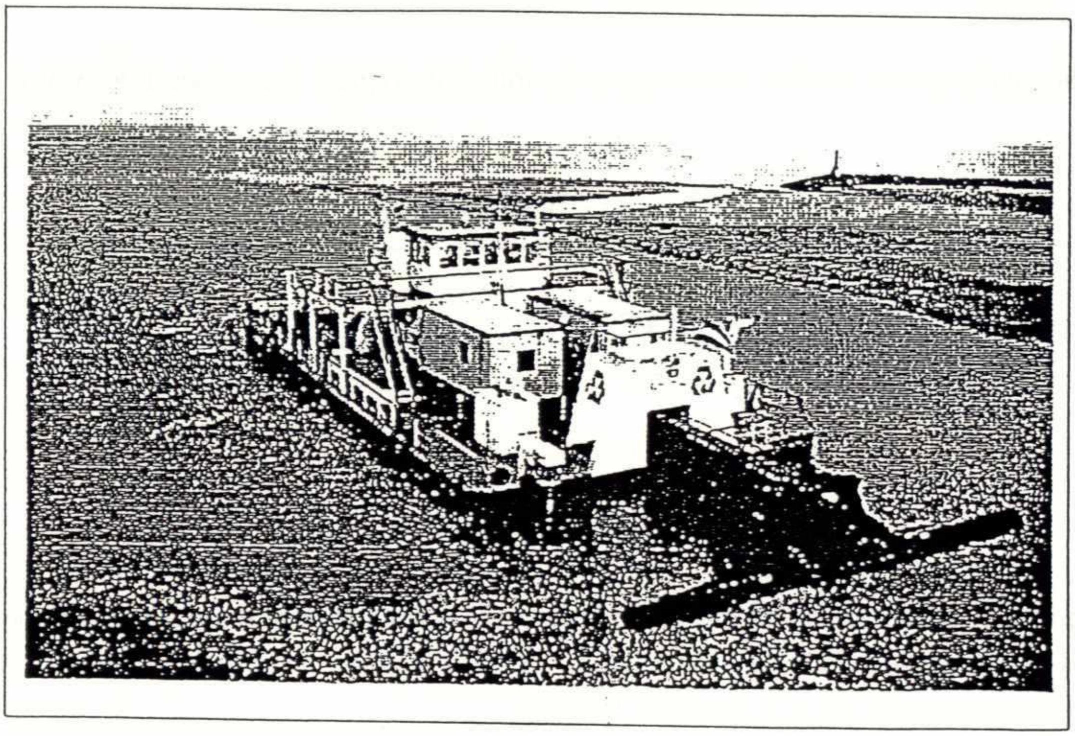

Fig. 3-29: The "Jetsed", (courtesy : Volker Stevin)
(Source: UN, ESCAP, 1989)

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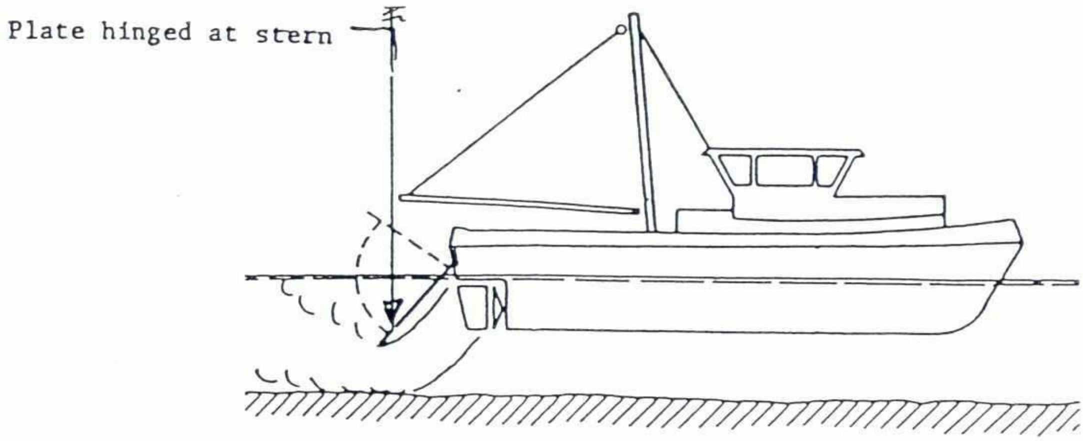


Fig. 3-30: Agitation by using ships propellers
(Source: UN, ESCAP, 1989)

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. 3-31):

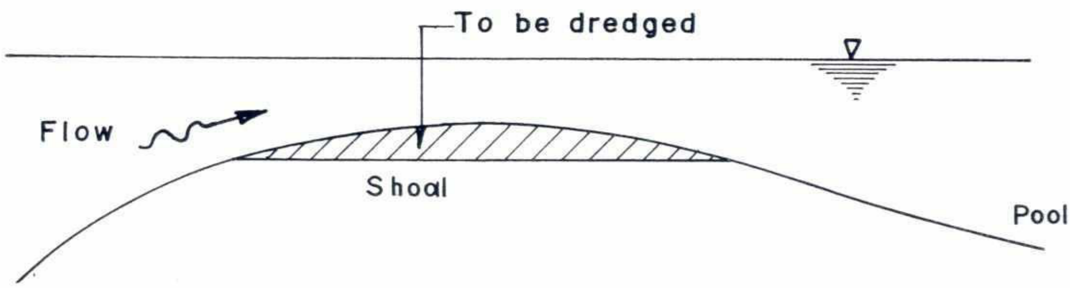


Fig. 3-31: Typical longitudinal profile

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, fasciae 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. 3-32)
- o bags or tubes (see Fig. 3-33)

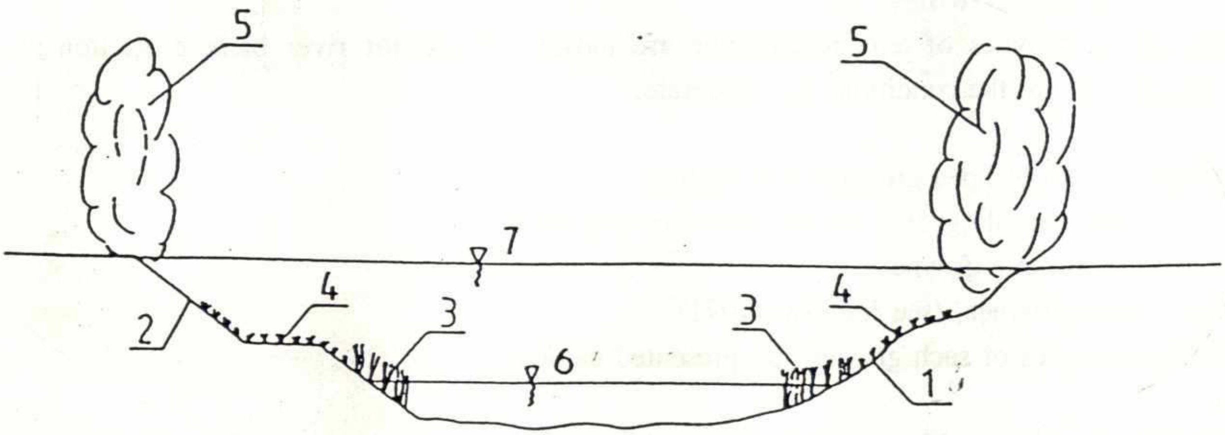


Fig. 3-32: Vegetation zones on a river bank. 1 - old bank, 2 - new composite profile. 3 - reed, 4 - grass, 5 - trees and shrubs, 6 -mean water level, 7 - high water level

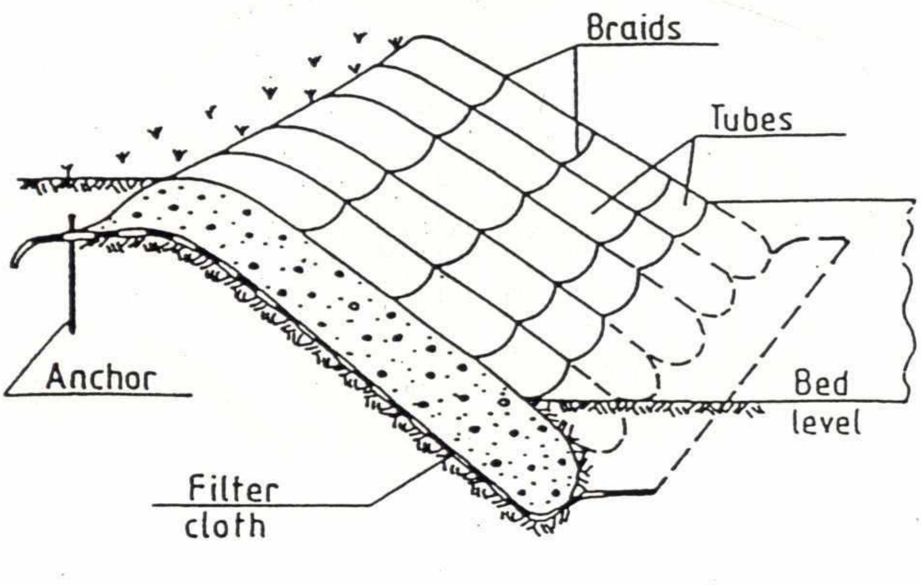


Fig. 3-33: Use of tubular gabions in bank protection
(Source: Pilarczyk et al., 1990)

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. 3-34) 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.

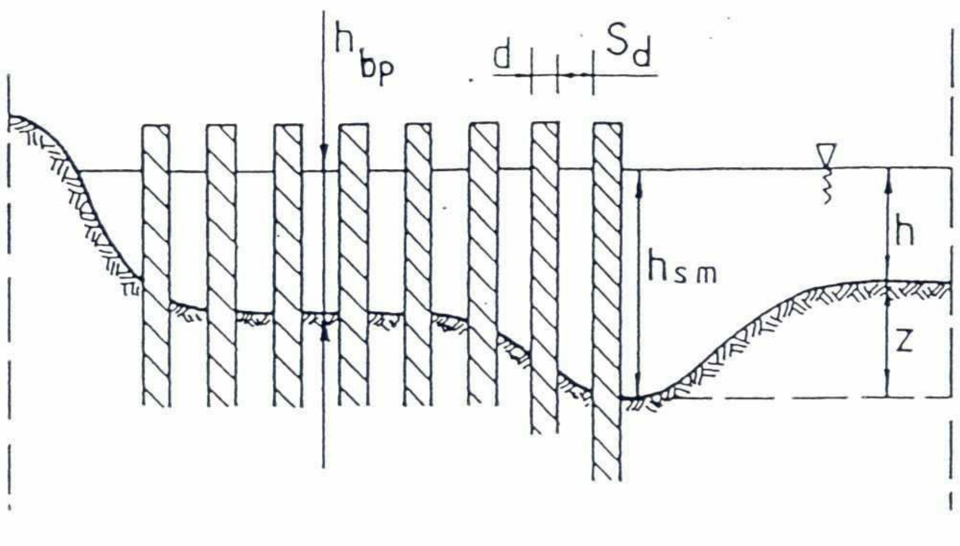
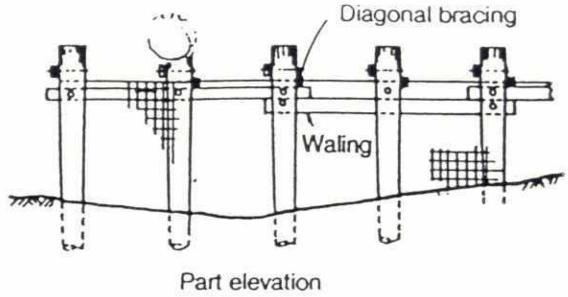
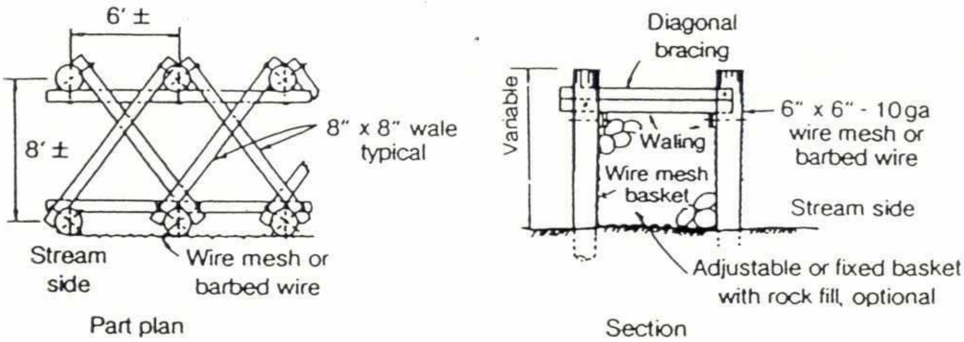


Fig. 3-34: Descriptive sketch of a permeable (needle) groyne (Source: Pilarczyk et al., 1990)

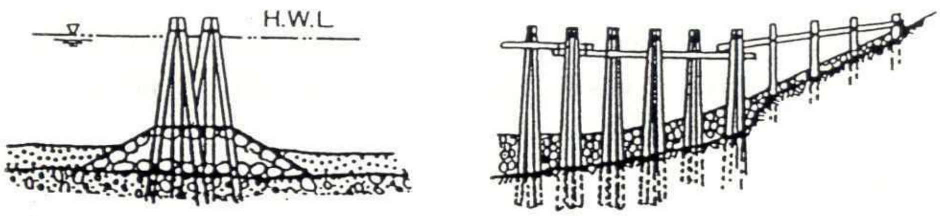
Examples of bamboo needle groynes are found in the Irawaddy River in Myanmar (Prins, 1990). For examples of timber pile dikes see for instance Chang 1988. Same examples of timber pile structures are also presented in Fig. 3-35.



(a) Single row timber pile with wire fence



(b) Double row timber piles with rocks and wire fence



(c) Pile clusters

Fig. 3-35: 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 this 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 are too high for these materials. A typical example of a timber spur in a smaller river (Gumti River) is given in Fig. 3-36.

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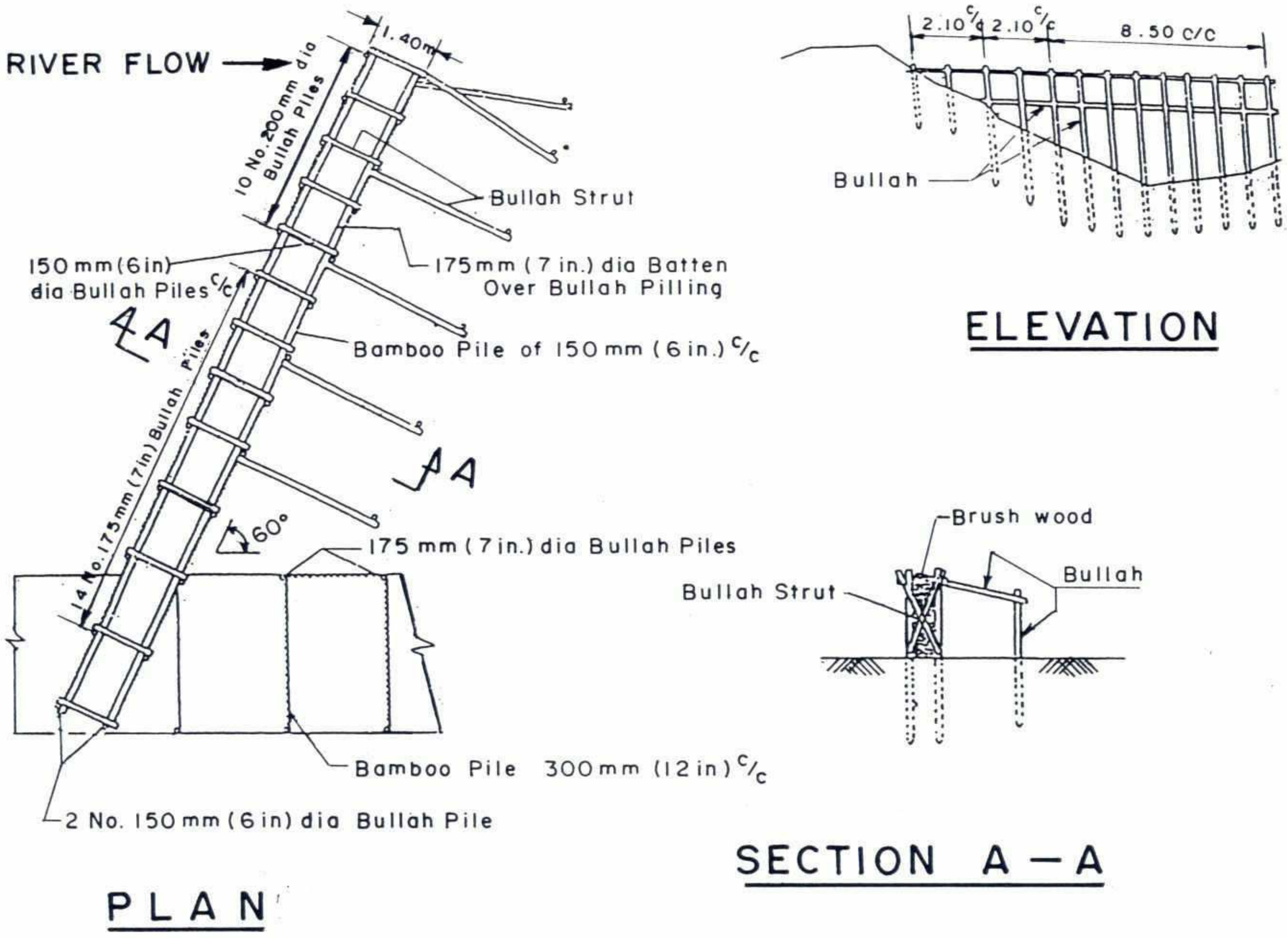

Fig. 3-36: Typical timber spur installed along Gumti river
(Source: Working Paper No.12, FCD III, BWDB)

(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. 3-37)

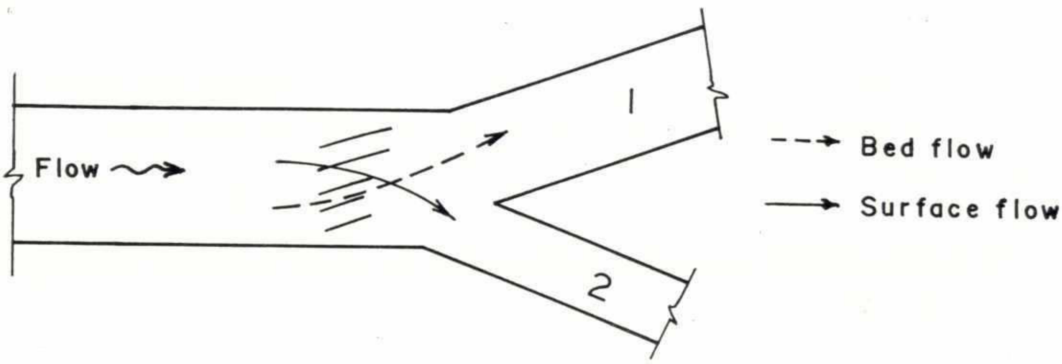

Fig. 3-37: Bottom vanes at a bifurcation

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

2) at a port entrance (see Fig. 3-38)

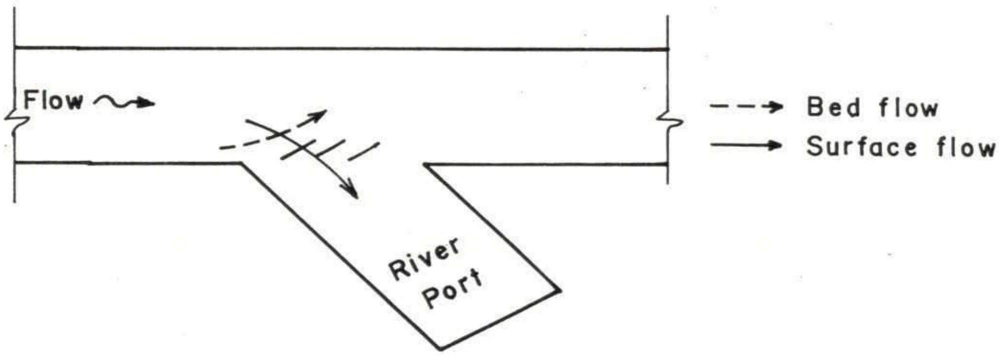

Fig. 3-38: Bottom vanes at port entrance

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

3) in a river bend

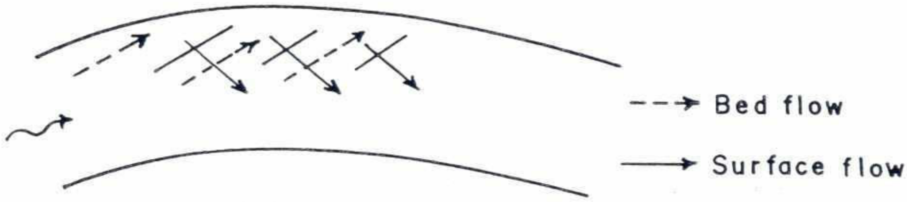

Fig. 3-39: Bottom vanes in river bend

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. 3-40.

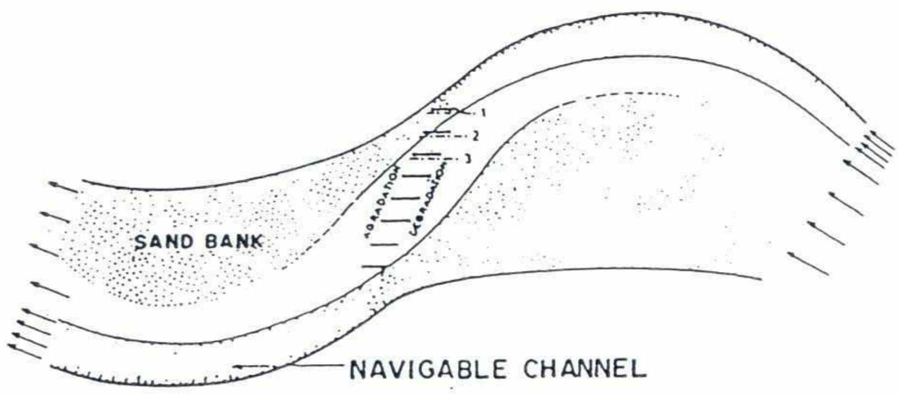

Fig. 3-40: Lay-out of bottom vanes across the river (Source: Jansen et al., 1979)

The principle and functioning of bottom vanes can be seen from Fig. 3-41.

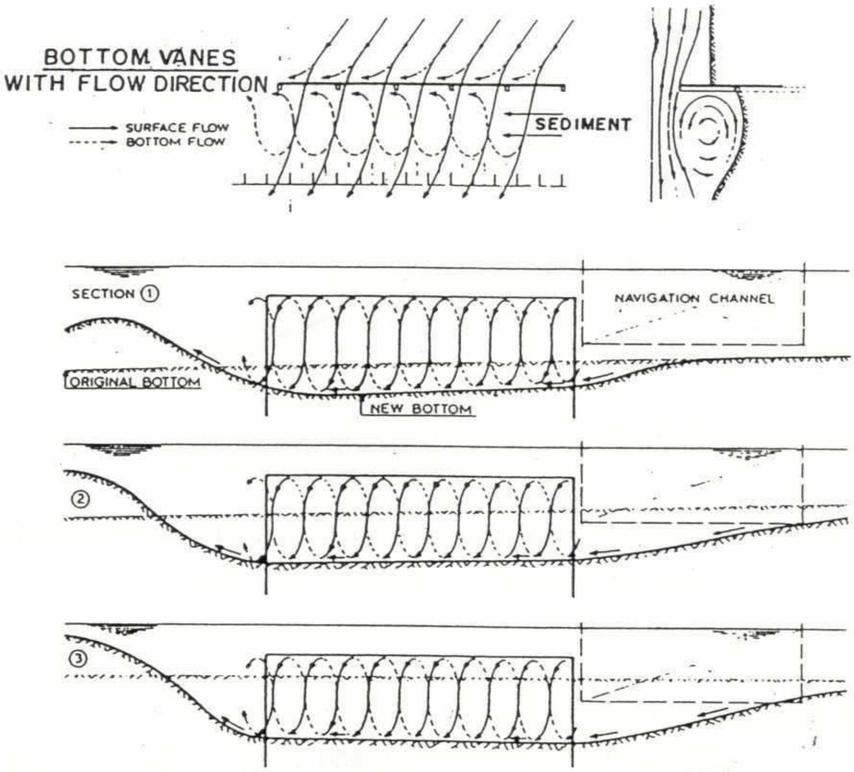

Fig. 3-41: Principles of bottom vanes in a river crossing (Source: Jansen et al., 1979)

5) along a river bank

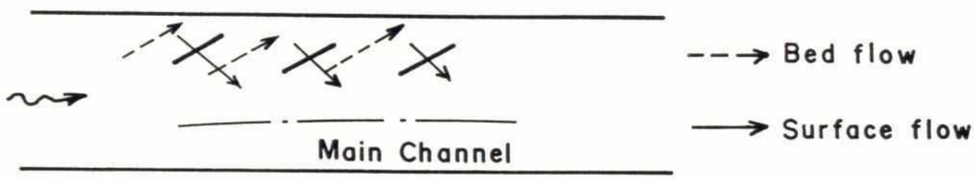

Fig. 3-42: Bottom vanes along river bank

Here (see Fig. 3-42) 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 are the screens in the Chao Phraya, 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. 3-43.

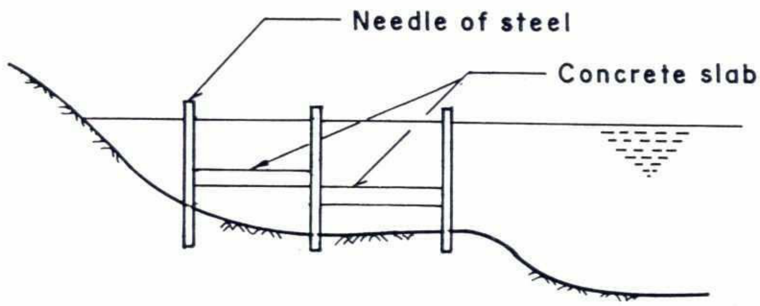

Fig. 3-43: Principle of Chao Phraya screens

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.3-44.

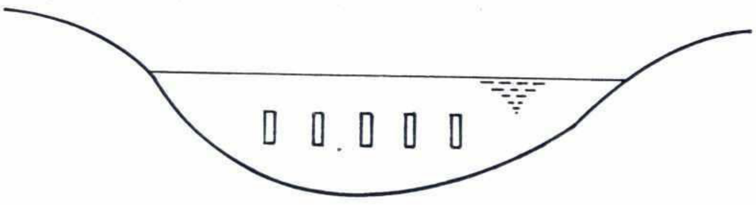

Fig. 3-44: Intermediate vanes in river (Potapov, 1950)

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).

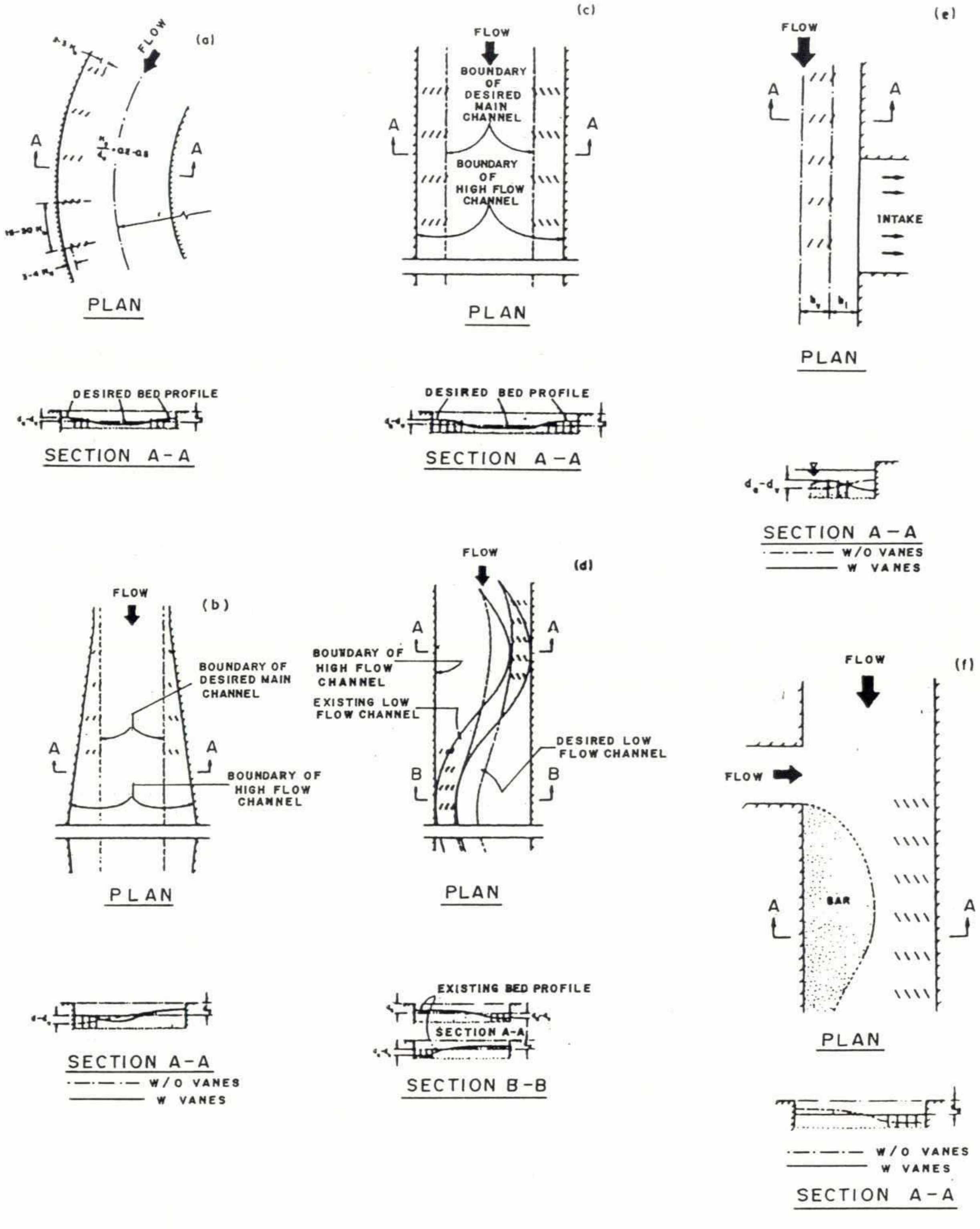

Fig. 3-45: Typical Vane Layouts for Stream-Bank Protection and/or Shoalling Control: (a) In Curved Channel; (b) In Widened Bridge Waterway; (c) In Navigation Channel or Bridge Waterway; (d) In Channel with Alternate Bars or Meanders; (e) At Water Intake; and (f) At River Confluence

(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. 3-46). 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. 3-47). 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.

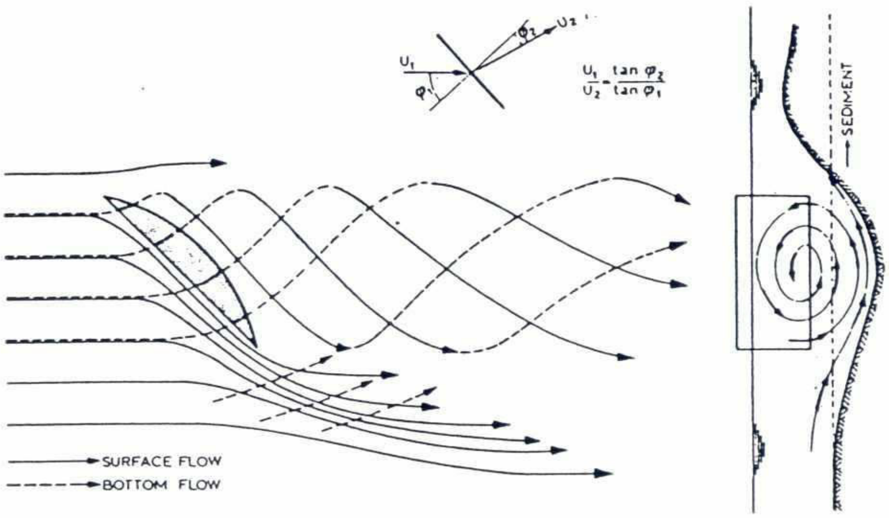

Fig. 3-46: The principle of a surface panel
(Source: Jansen et al., 1979)

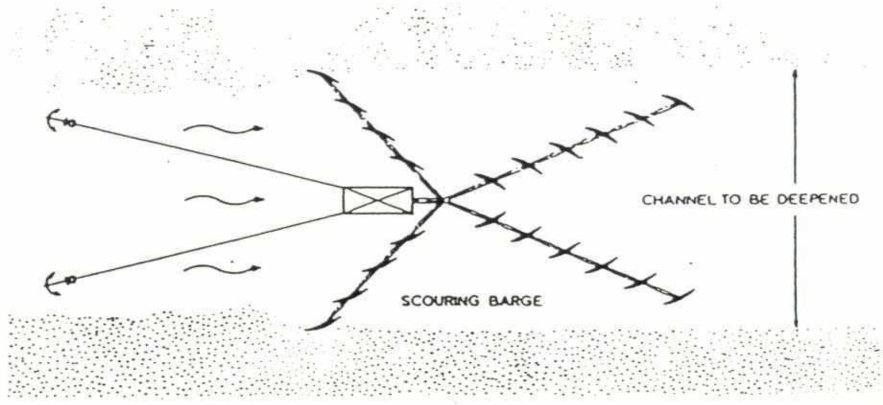

Fig. 3-47: Surface panels united in a 'scouring barge'
(Source: Jansen et al., 1979)

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. 3-47).

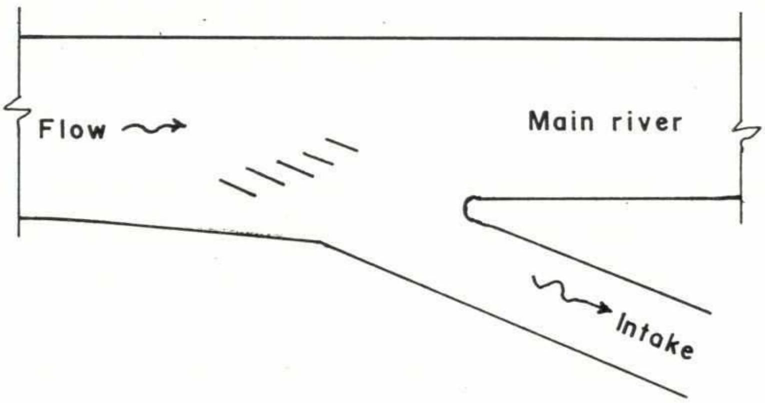

Fig. 3-48: Floating vanes before intake

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.

o Fixed surface vanes

Fixed surface vanes like bandals are applied for various purposes (see Fig. 3-49). Two important objectives are:

- o fairway improvement
- o closing of secondary channels

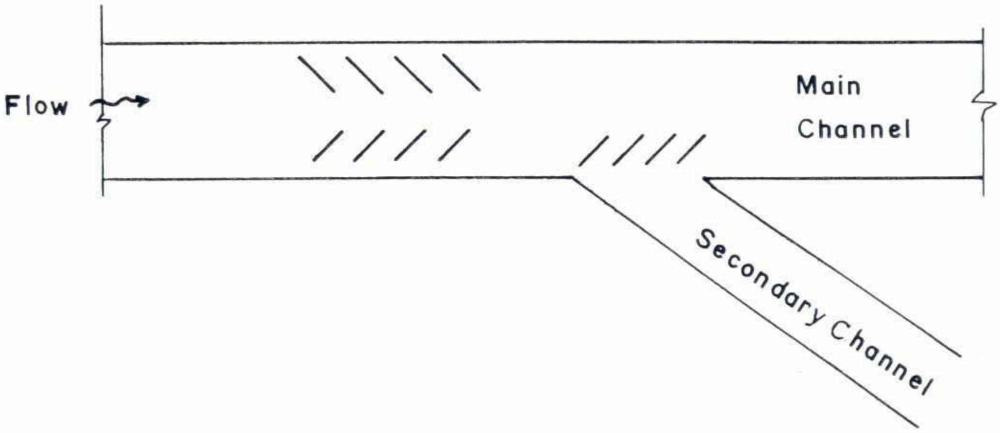

Fig. 3-49: Application of bandals

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.

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. 3-50.

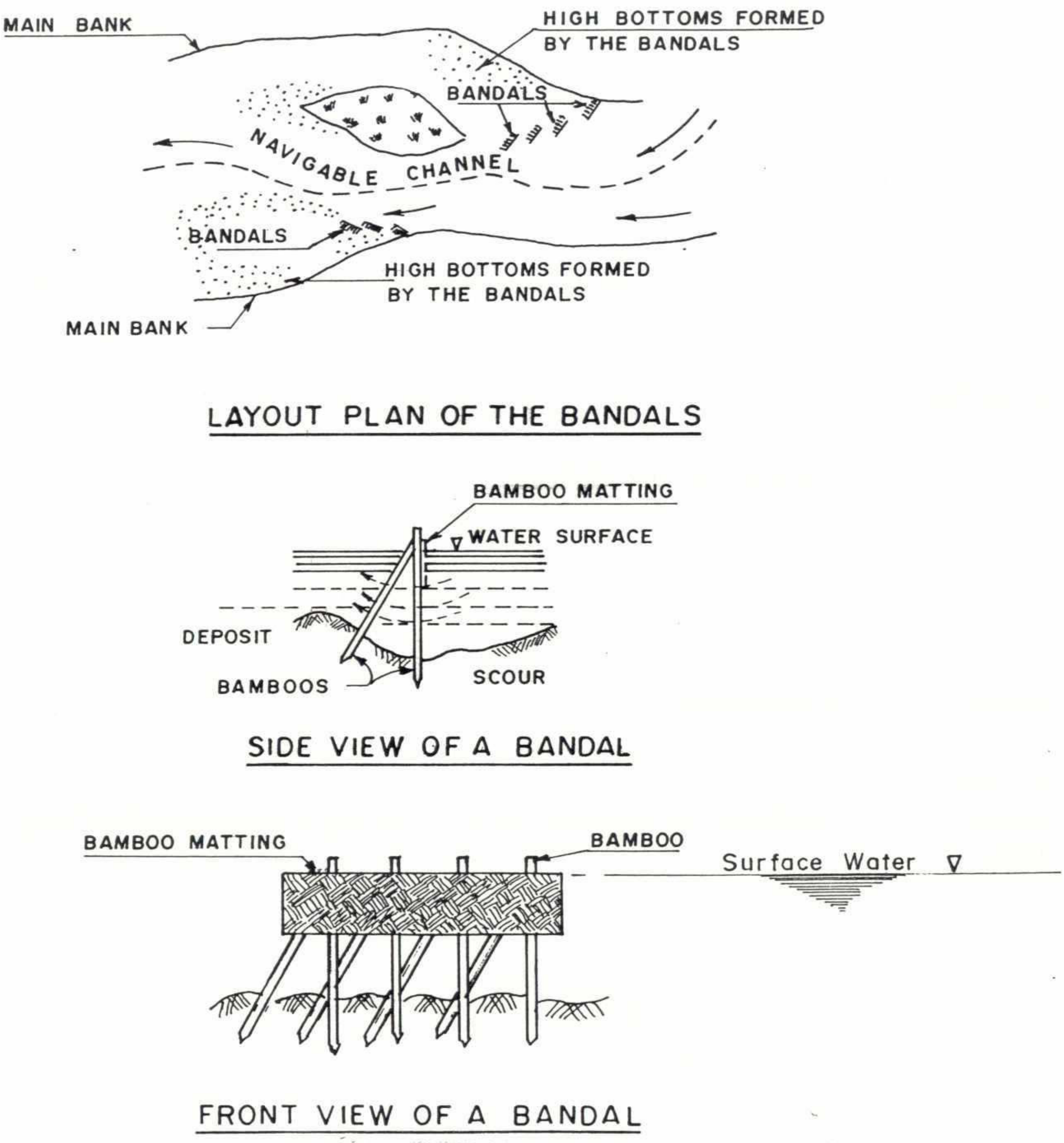

Fig. 3-50: Plan and Elevation of bandal
(Source: Nishat, A., 1986)

In reality the construction of the bandals and the process of erosion is some-what 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 bandall 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

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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.

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. 3-51) and steel jacks (see Fig. 3-52) are given by Chang, 1988.

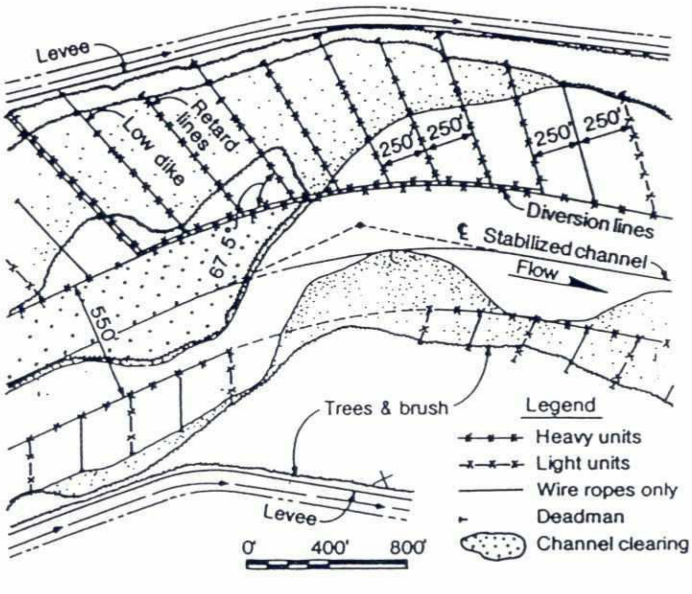

Fig. 3-51: Typical jetty field layout - Rio Grande
(Source: US Army Corps of Engineers in Chang, 1988)

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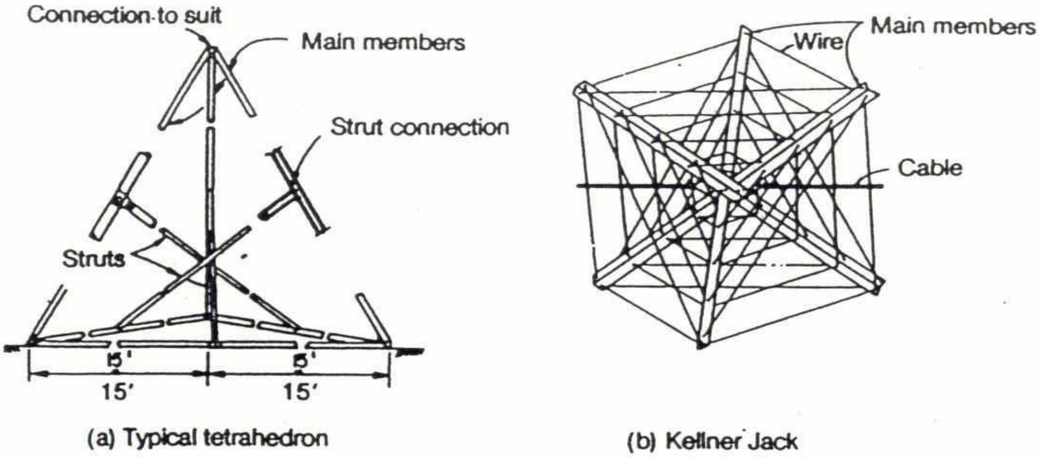

Fig. 3-52: Steel jack
(Source: Chang, 1988)

(ii) Cows
For arrangements of cows reference is made to Fig. 3-53 and 3-54.

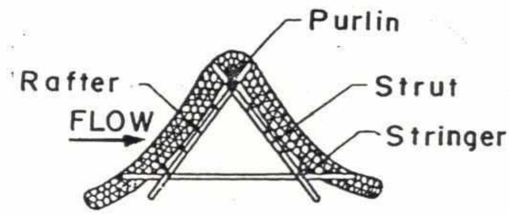

Fig. 3-53: Stabled Cow
(Source: UN, ECAFE, 1953)

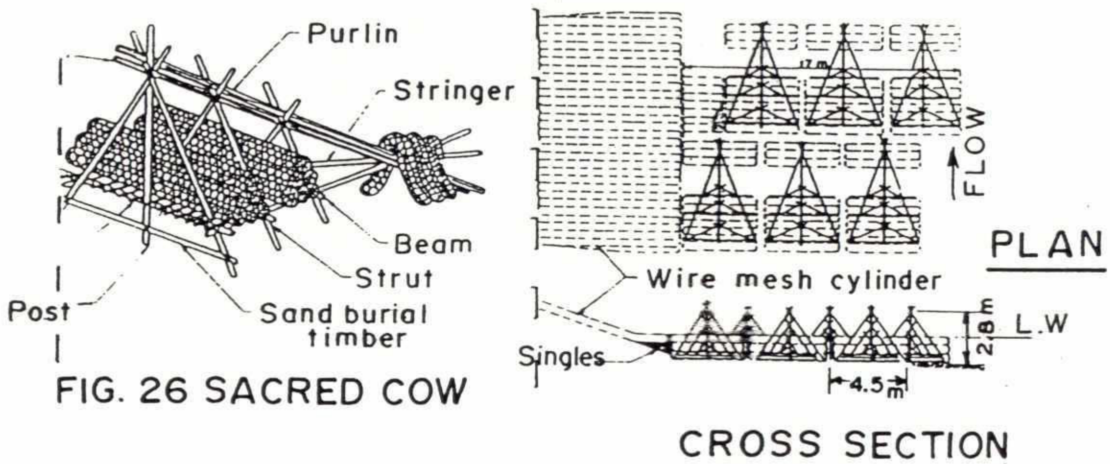

Fig. 3-54: Arrangement of sacred cow
(Source: UN, ECAFE, 1953)

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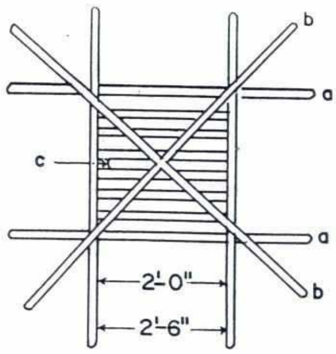
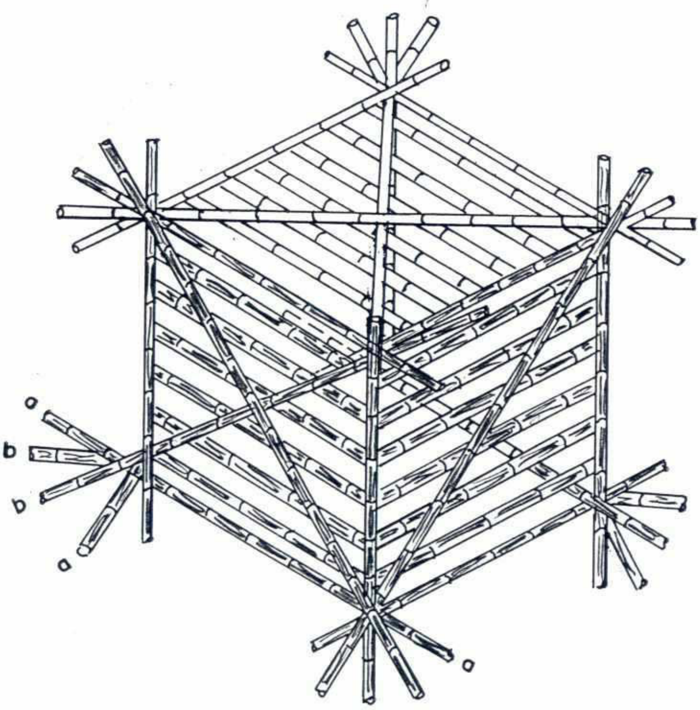
(iii) Porcupines

Porcupine is a kind of frame made from bamboo and filled with brick bats. Porcupines are placed on river bank 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 river as has been found in Satkhira and other places in Bangladesh.

Their use in flushy river 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. 3-55.



PLAN AND END ELEVATION OF PORCUPINE



ISOMETRIC VIEW OF PORCUPINE

Fig. 3-55: Porcupine
(Source: BWDB Typical Drawing)

(iv) Retards

Where the flow velocity is moderately high and the river carries silt, "Retard" 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.

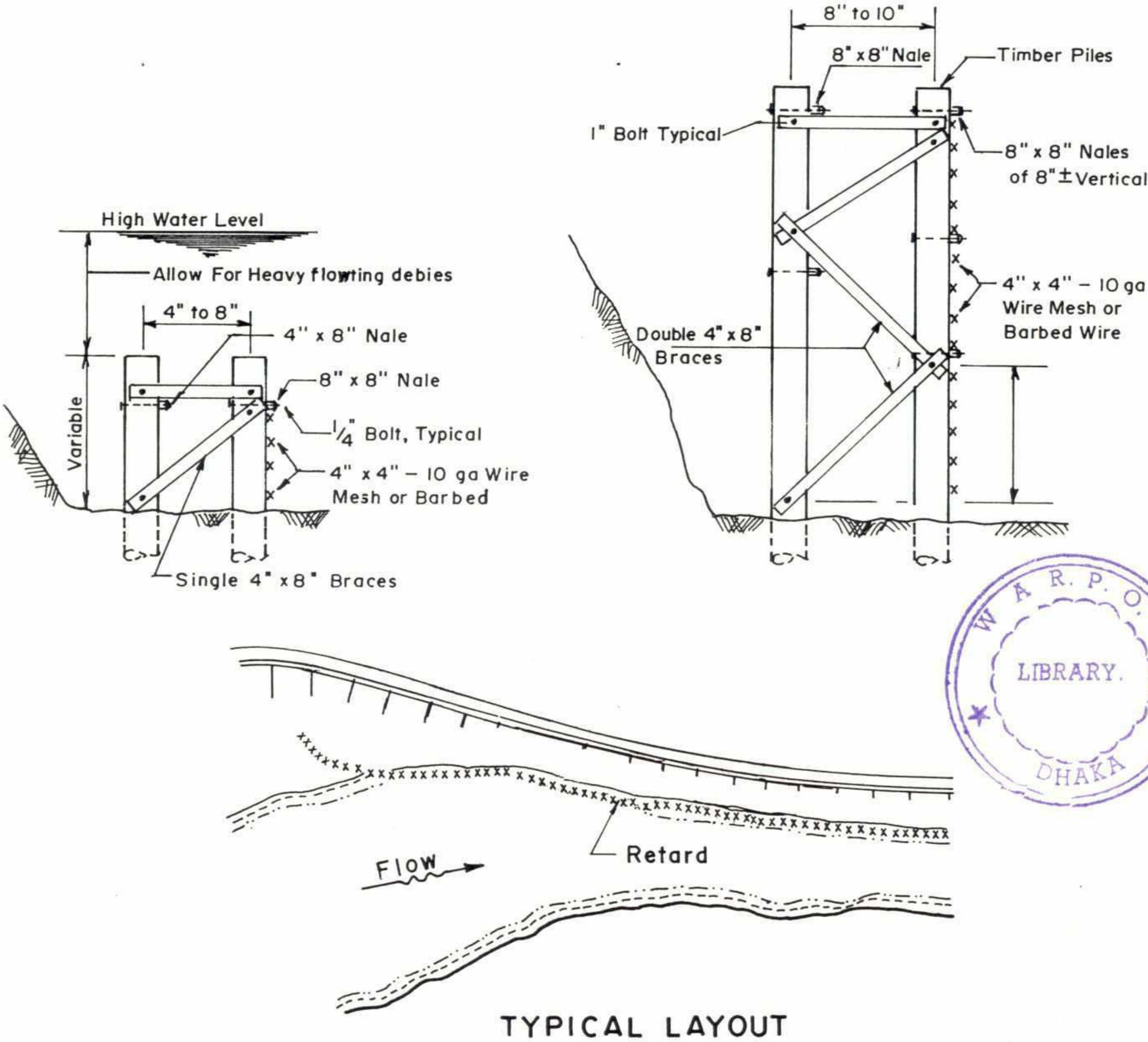


Fig. 3-56: Bank and shore protection in California Highway Practice (Source: Highway Division, Deptt. of Public works. California 1970)

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 being a subsection of flood 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 (China)
- Mississippi River (United States)

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

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 River Rhine

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. 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 administrative strategies could be developed.

The main objectives of river training were appropriate distribution of flow in the Rhine delta (The Netherlands) 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. 4.1 and 4.2 (Petts, 1989).

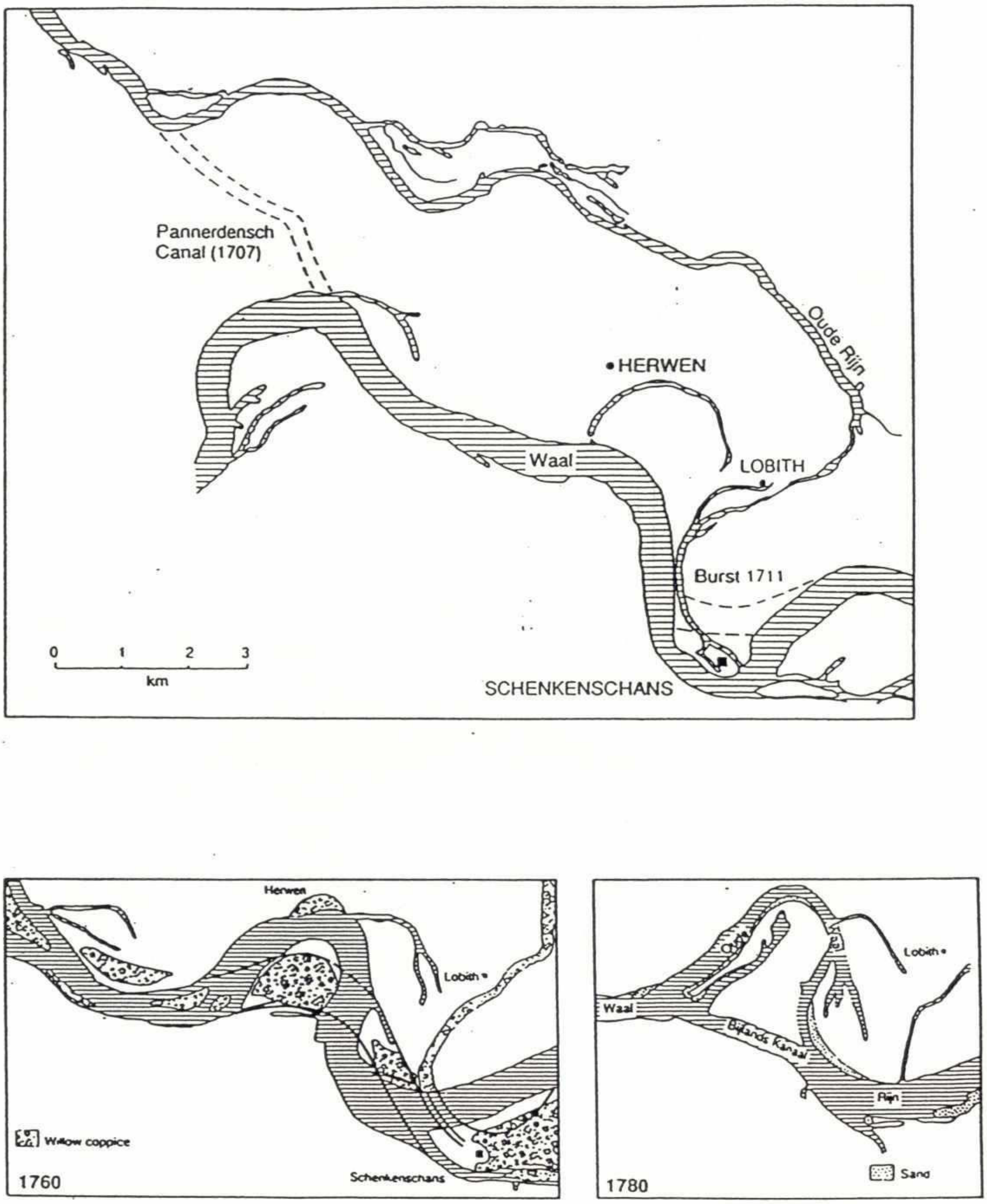


Fig. 4-1: The situation near the first Rhine bifurcation at the end of the 17th century above, and, below, the Waal meander near Herwen around 1760 (left) also showing the situation after the construction of the Bijlandsch Kanaal around 1780.
(Source: Petts et al., 1989)

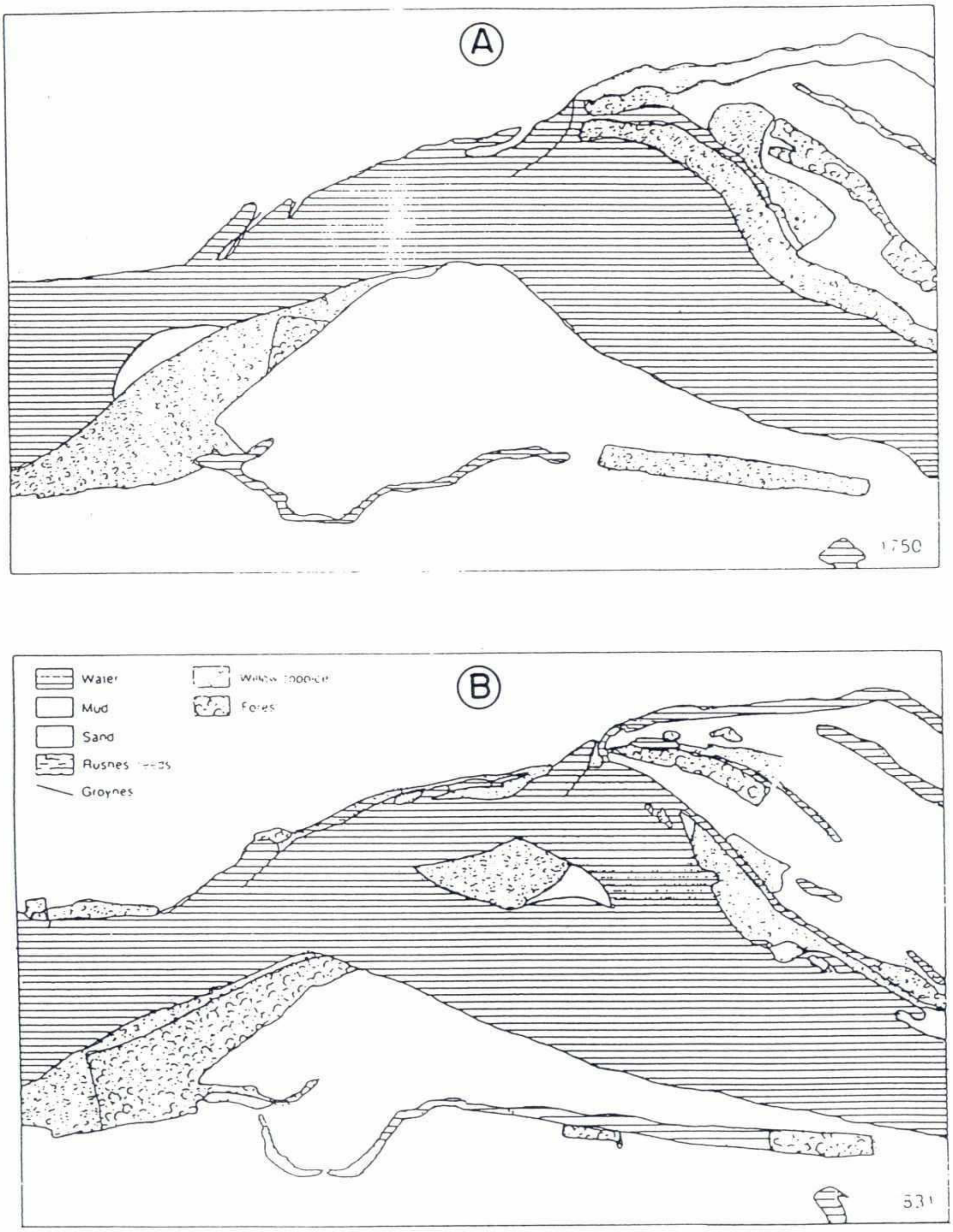


Fig. 4-2: The Waal near river km 899-901 showing channel changes over a period of about 200 years. About 1830 (Fig. B) the main channel had been diverted southward, a sand bar had developed between the two channels, and some dyke sections were protected by groynes. By 1870 (Fig. C) the northern channel had been dammed. The dam is protected by short groynes; and

(Source: Petts et al., 1989)

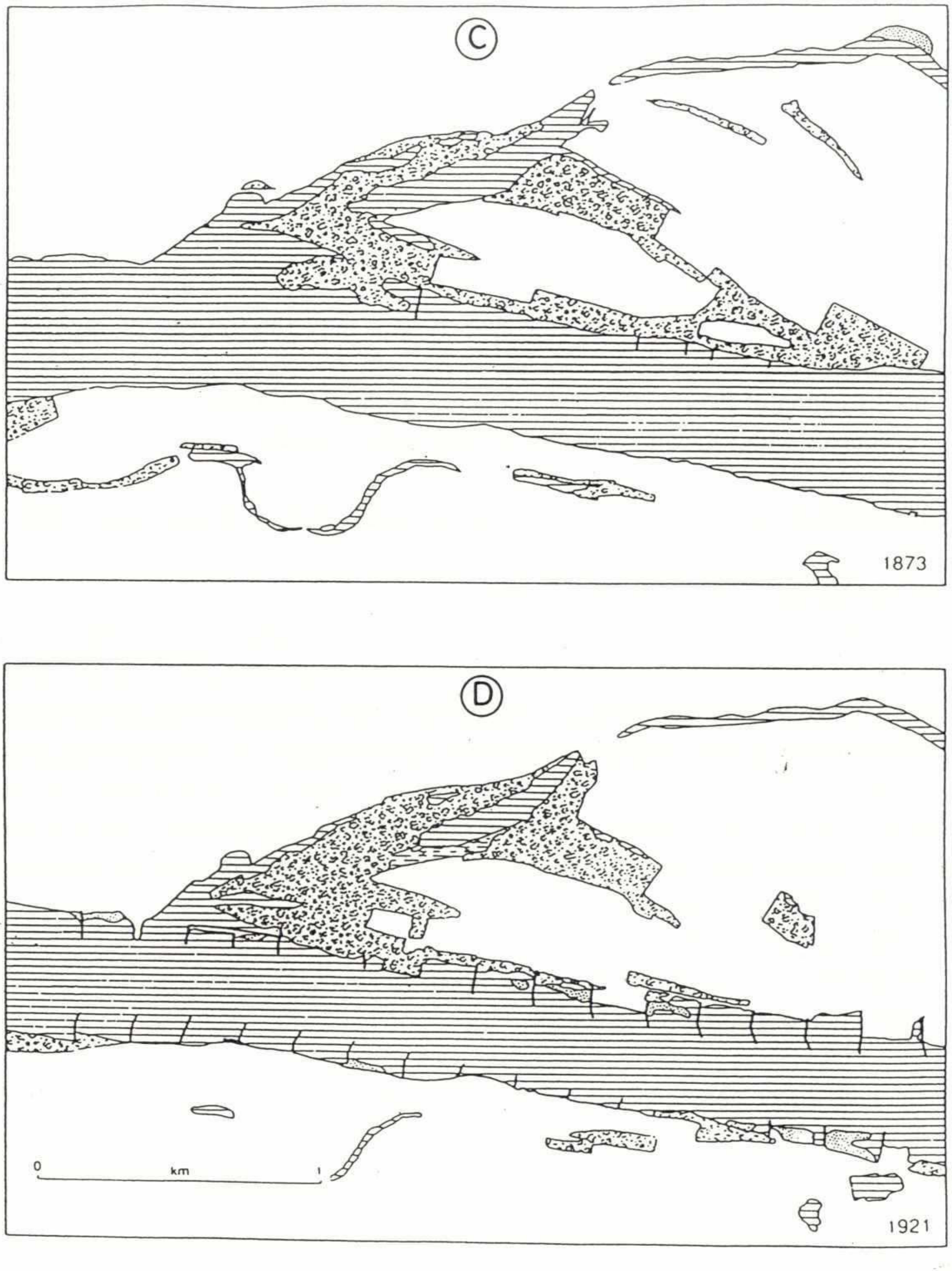


Fig. 4-2 (continued): The southern bank has been straightened. Behind the dam, large areas have been silted up and are occupied by willow coppices. By 1920 (Fig. D), channel constriction by groynes had been completed. The open water area had decreased and older willow coppices had been removed, but new willow coppices had developed in areas recently silted up.

(Source: Petts et al., 1989)

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 upto 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.

o River Rhône (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 when 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 River 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. 4-3.

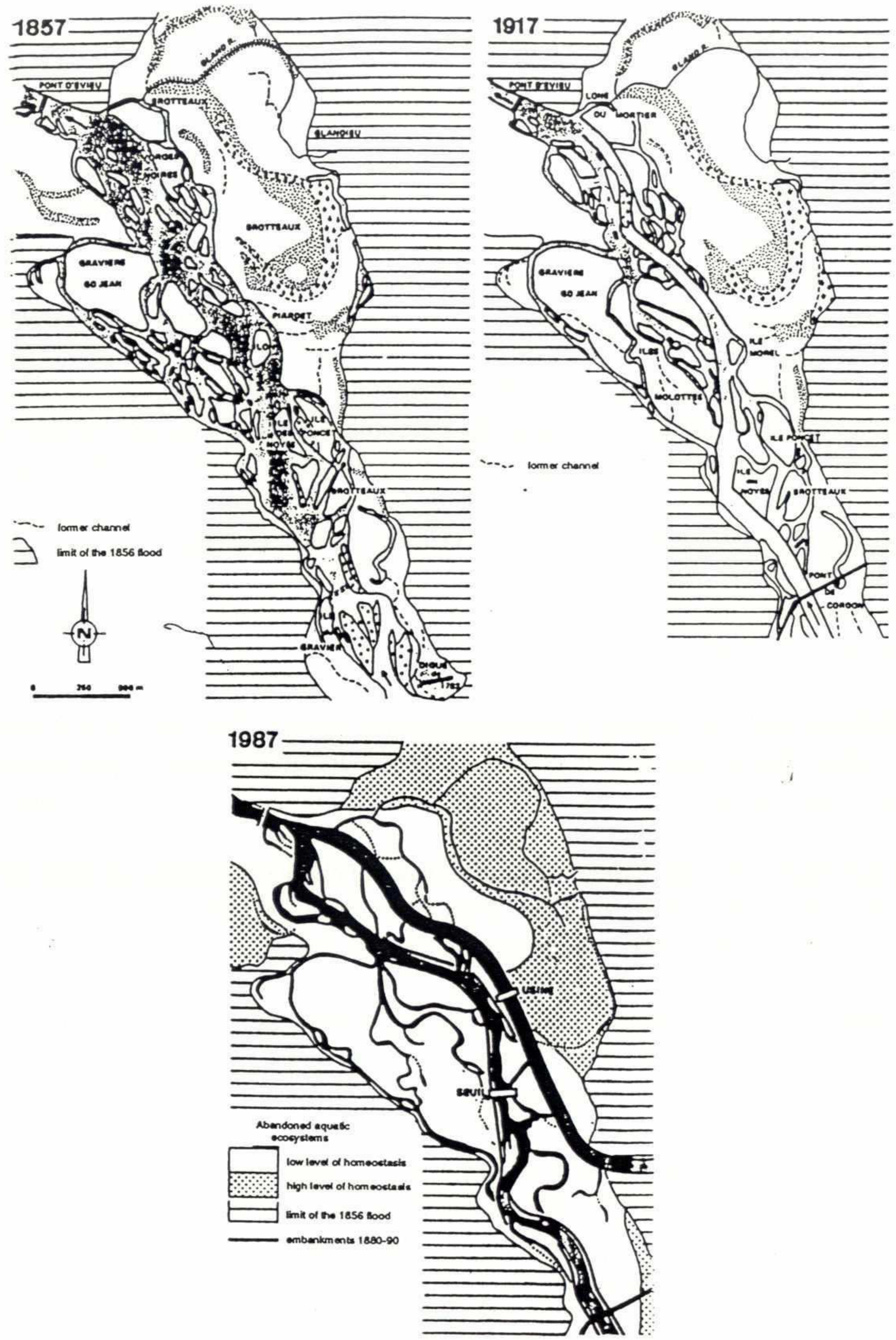


Fig. 4-3: Lay-outs of the Brégnier-Cordon sector showing the numberous braided channels within the aggrading central area and the anastomosed channels in the lower lateral belts; in 1850 before the civil engineering works; in 1900, after the construction of the submersible embankments; and in 1987, after the hydroelectric equipment construction. (Source: Petts et al., 1989)

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o **Lower Weser River (Germany)**

The main objective to channelize the Lower Weser River in Germany is 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. 4-4). 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.

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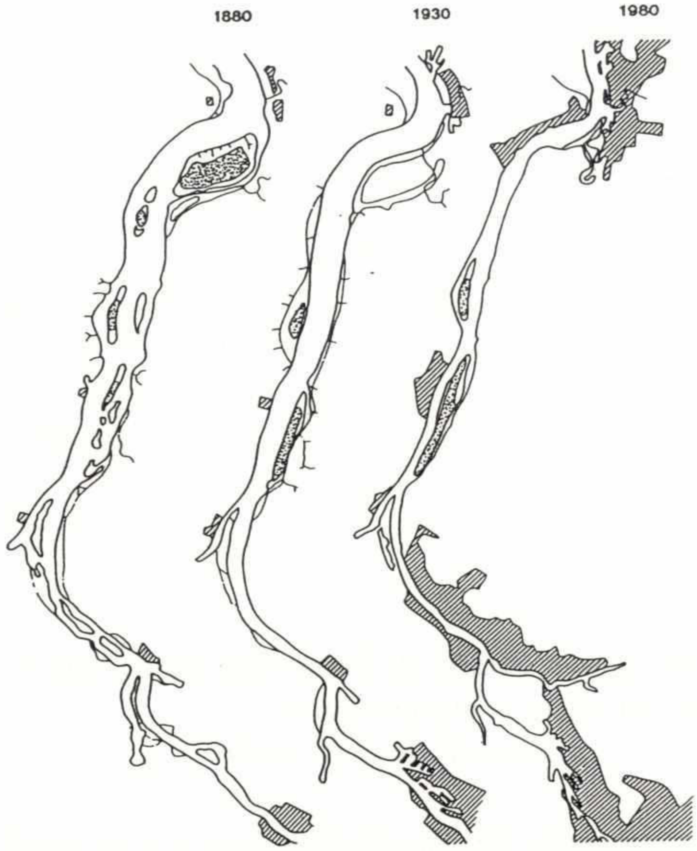

Fig. 4-4: Reduction of the Lower Weser and after channelization around 1880, 1930, and 1980.

(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. 4-5).
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. Disposal sites were located in adjoining marshes behind the dykes and, thus, changed the ecological conditions there drastically.

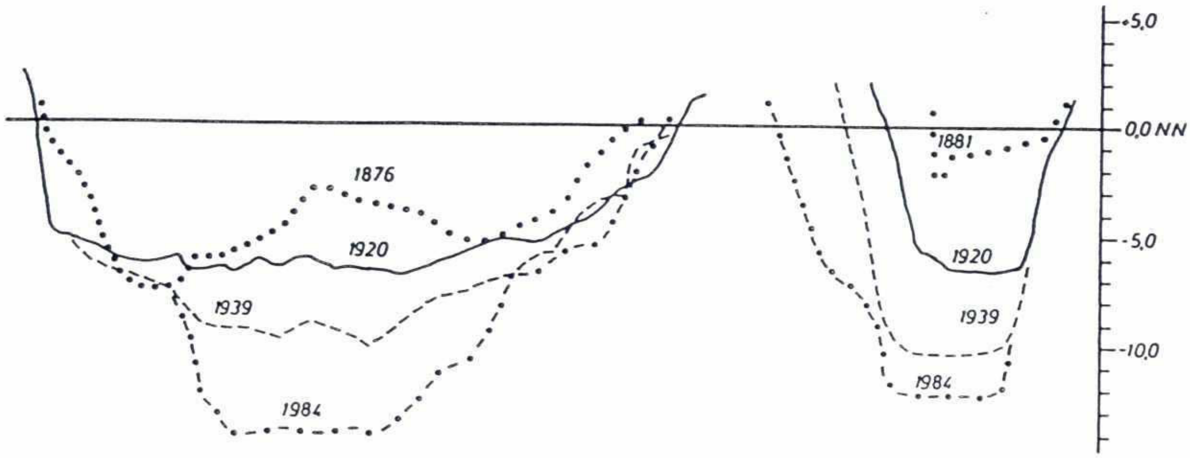

Fig. 4-5: Profiles of the Lower Weser at Brake (km 405.7) and Bremen Harbour (km 372.7) during different stages of channelization.
(Source: Petts et al., 1989)

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

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

To make use of the experience gained in other countries on comparably large braided river participants of GOB and the Consultant made a study tour to the Yellow and Yangtze kiang Rivers in China and to the Mississippi in USA. The impressions obtained by own experience and the information gathered in intensive discussions with representations 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.

Additionally the main findings and conclusions are summarized below.

Experiences from the Yellow and the Changjiang River

- Long history of human interventions in the natural behaviour of mighty rivers of Yellow and Changjiang are available. Such knowledge suggests that flood control measures are slow processes of learning the river and gradual process of adjustment: a truly trial and error approach towards taming the river. Any major interventions in a big river might have very adverse effects.

- Experiences from Yellow and Changjiang Rivers suggest that no natural "Nodal Point" in a river appears do exist, which may be termed as a "hard point" 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, Jamuna offers a much difficult problem to solve than either of the Yellow or Changjiang. 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. 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.

- 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.

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, of selection made and presented is 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. For the time being, this measure will not be considered with respect to the preliminary selection.

5.1 MEASURES AT BIFURCATION

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 the previous Sections 2.4.1.

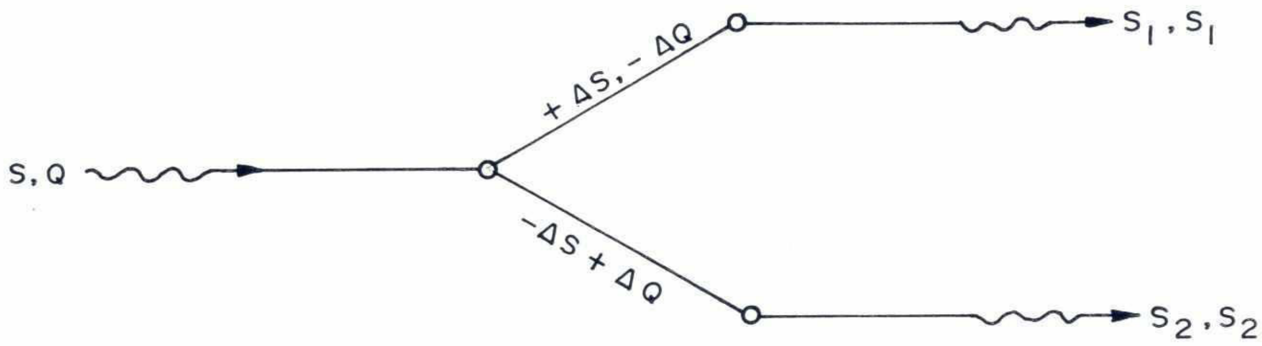

Fig. 5-1: Notation at bifurcation

Notes :

- 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 rations 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.

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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

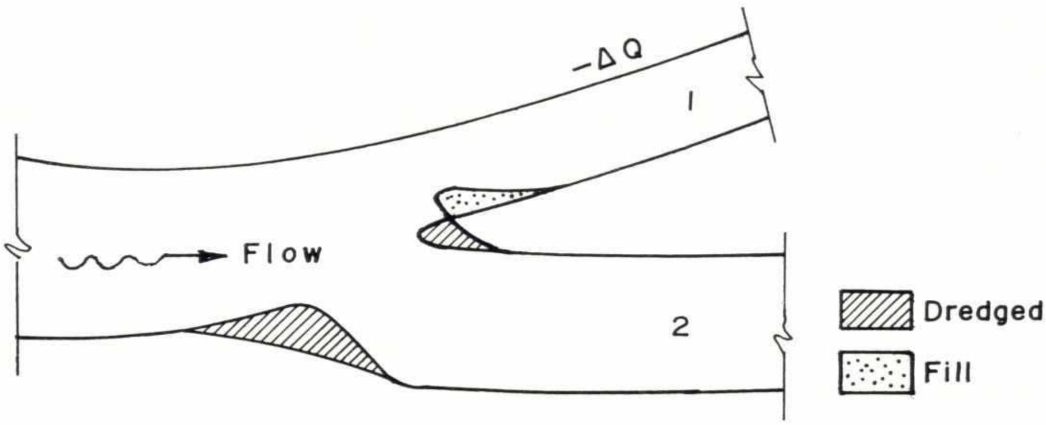

Fig. 5-2: Planform corrections

The principle of this method (see Fig. 5-2) 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

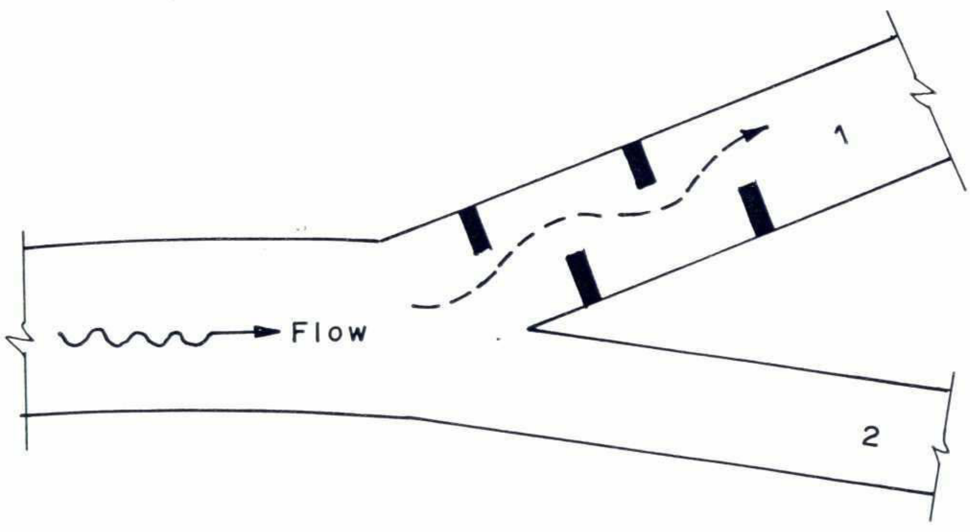

Fig. 5-3: Groynes in outer channel

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. 5-3) 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:

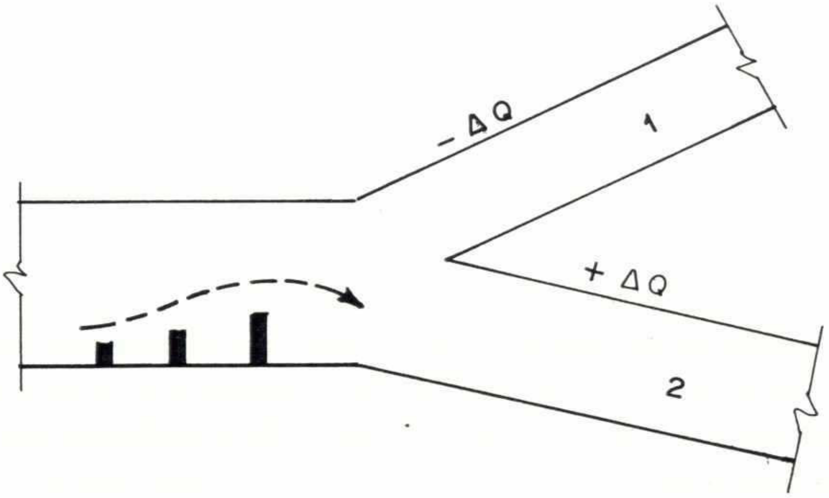

Fig. 5-4: Groynes in the upstream channel

The aim of the lay-out as presented in Fig. 5-4 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 scream head is avoided.

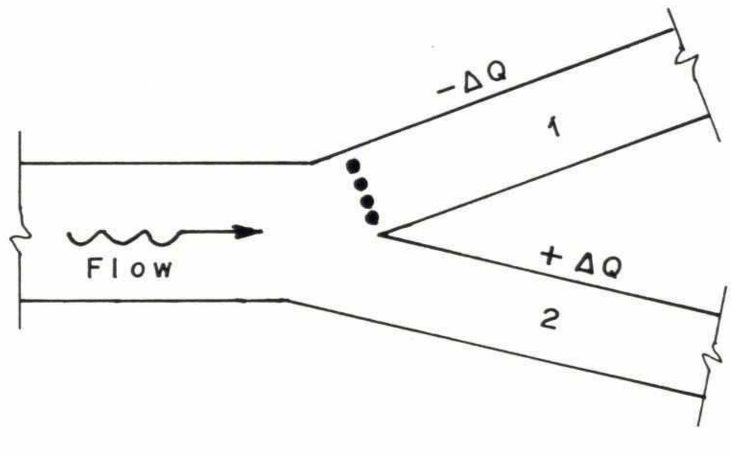

Fig. 5-5: Open pile screen

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 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) depends 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. 5-6 in the Yangtze Kiang.

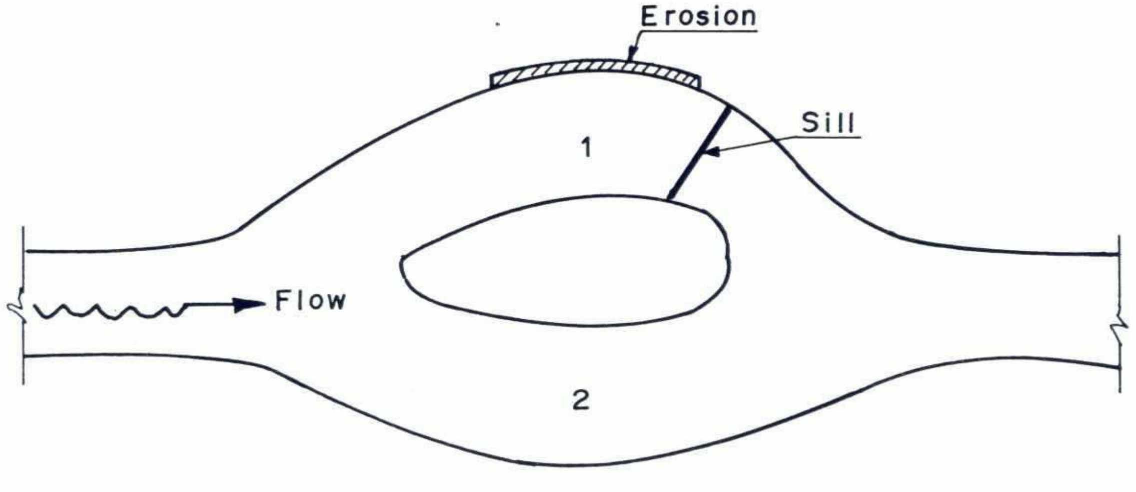

Fig. 5-6: Sill

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 Vanes

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

(i) Bottom Vane

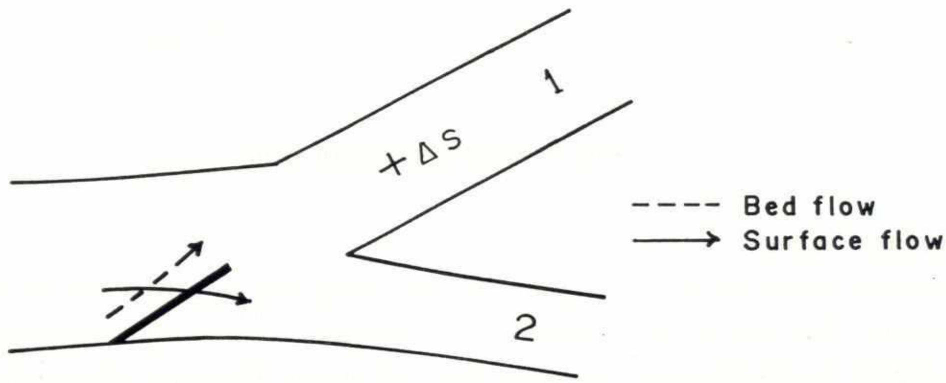

Fig. 5-7: Bottom vane

The bottom vane (or panel) (see Fig. 5-7) 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 is 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. 5-8).

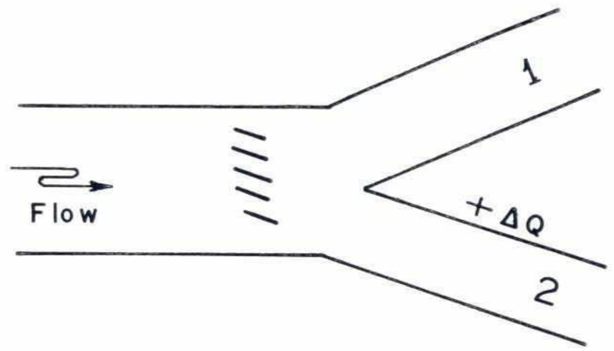

Fig. 5-8: Intermediate vane

(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. 5-8 above. However, the bandal can also be placed before the entrance of channel 1, as indicated in Fig. 5-9.

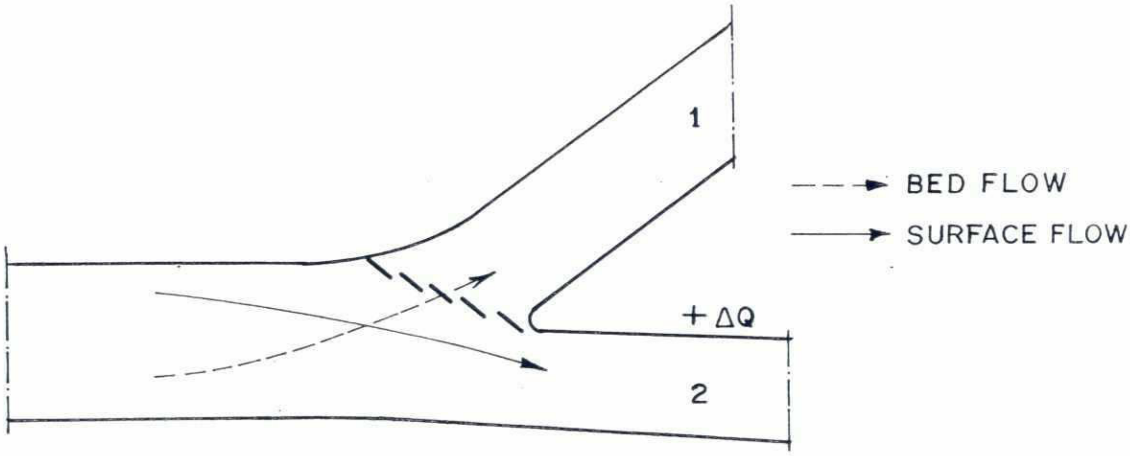

Fig. 5-9: Bandal

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 CHANNEL

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. 5-10) with the deepest part near the eroding outer bank can be changed (channel 1 is supposed to be curved) :

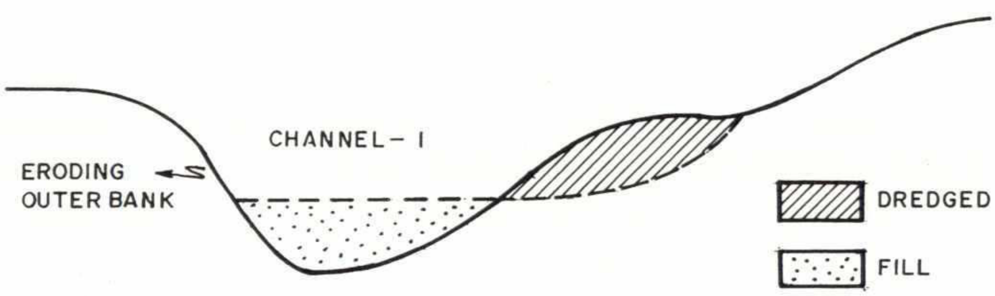

Fig. 5-10: Changing cross-profile

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The reshaping will have the right effect on the horizontal flow distribution: the velocities near the outer bank will reduce and the velocities near the opposite bank will increase. This type of corrections 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. 5-11).

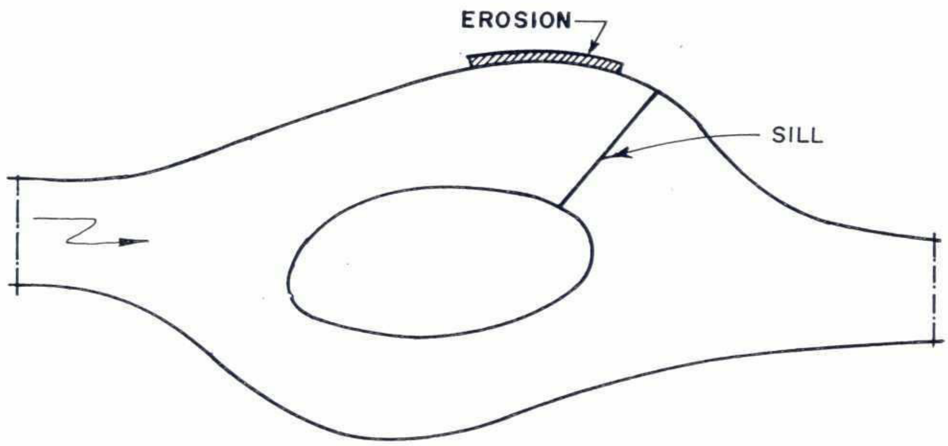

Fig. 5-11: Sill in outer channel

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 reduce, reducing the scour.

5.2.3 Vanes

Some possibilities to reshape the cross-section in the outer channel using vanes are indicated below.

(i) Bottom Vanes

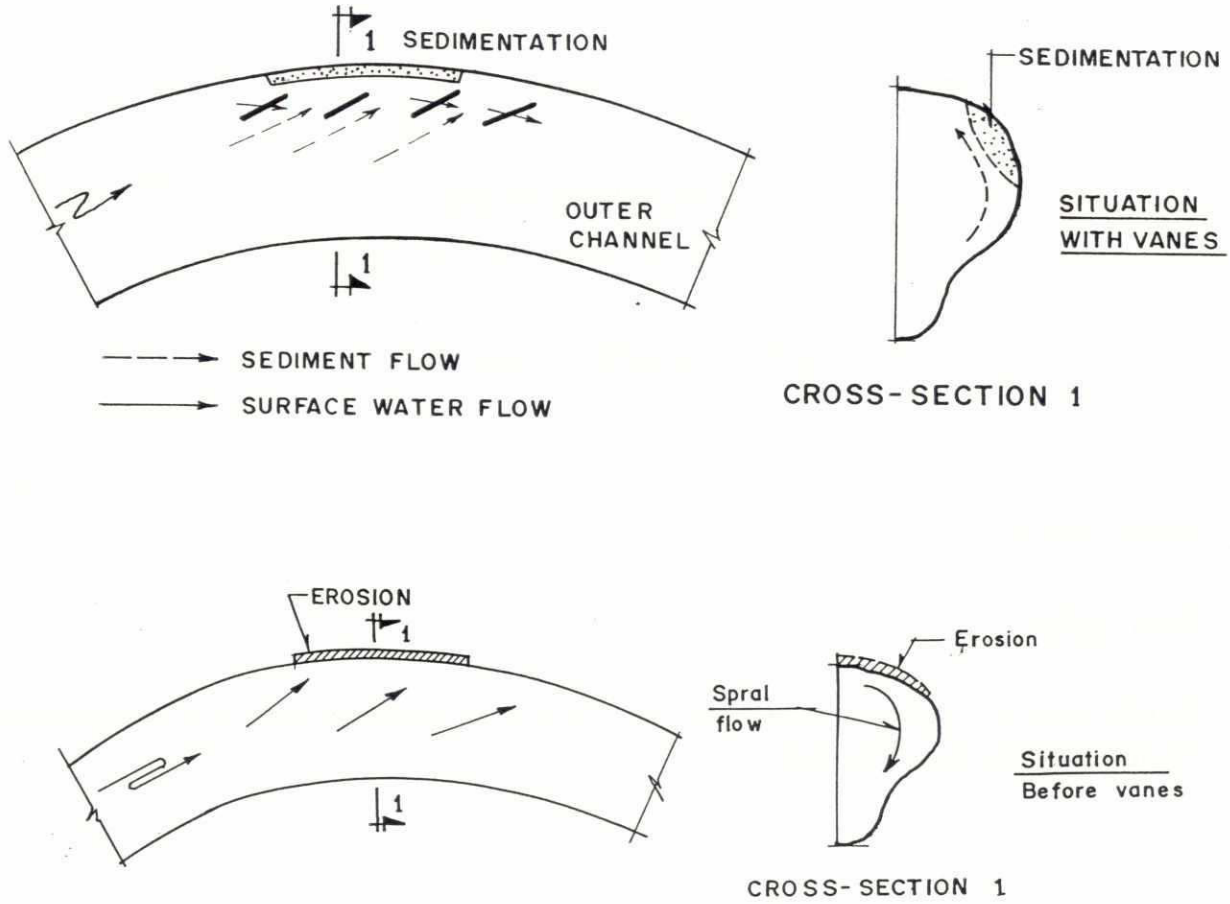

Fig. 5-12: Bottom vanes in outer channel

The bottom vanes (see Fig. 5-12) are catching the bed load transport, directing it towards the outer bank. The surface flow, tending to cross in the vanes perpendicularly, is directed from the bank. Is it possible to install these vanes and arrest with them a part of the dunes passing by ?

(ii) Surface Vanes

Obviously also bandals or floating vanes can be applied to adjust the bed flow and surface flow (see Fig. 5-13).

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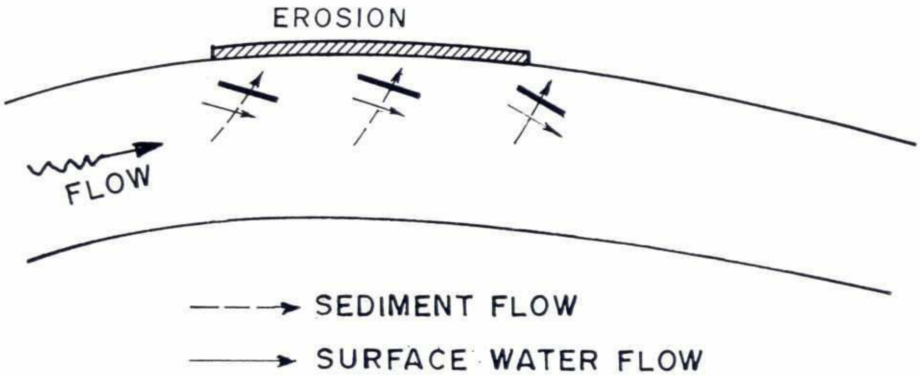

Fig. 5-13: Bandals in outer channel

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.

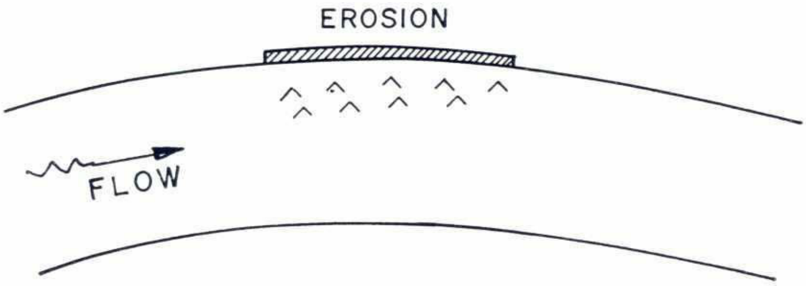

Fig. 5-14: Jacks near the outer bank

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. 5-15).

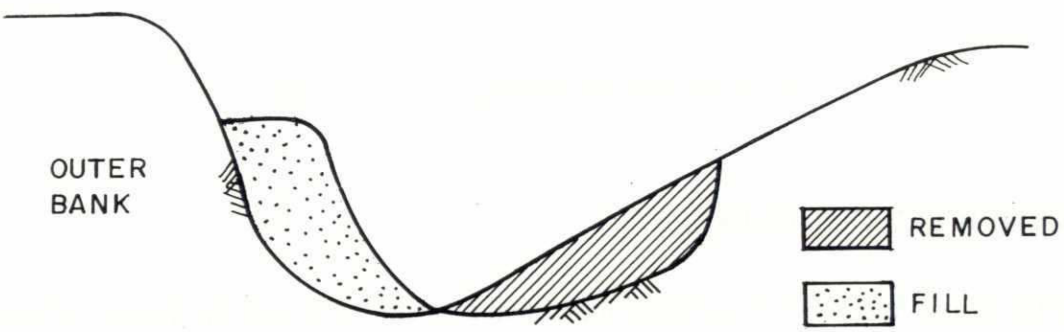

Fig. 5-15: Bank protection by suppletion

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 length of 500 m, depth of 20 m and width of 1000 m. This should lead to a dredging volume of 10 x 10^6 m^3 which is much more than the available dredge capacity (see Table 3-3). 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.

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5.3.3 Permeable Groynes

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

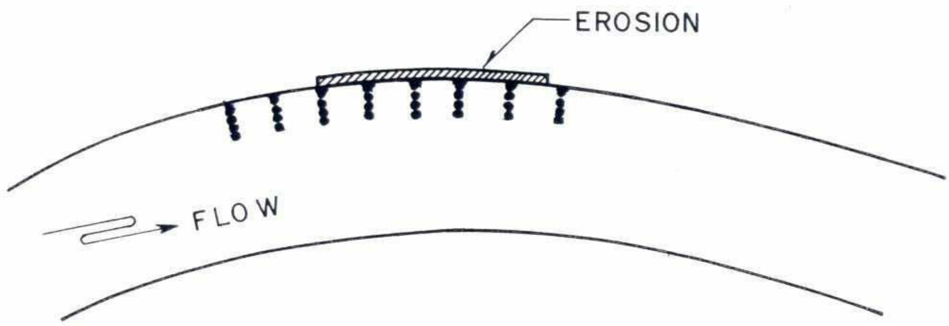

Fig. 5-16: Needle groynes at outer bank

5.4 ARTIFICIAL CUT-OFFS

This measure has to be considered as an individual measure with specific features of possible interest for AFPM. This 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 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:

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$$\tau \geq \tau_{critical} \tag{5.1}$$

where τ = shear stress during flood, and $\tau_{critical}$ = 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:

$$\tau = \rho g h_{floodplain} \Psi i \tag{5.2}$$

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. 5-17 a 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.

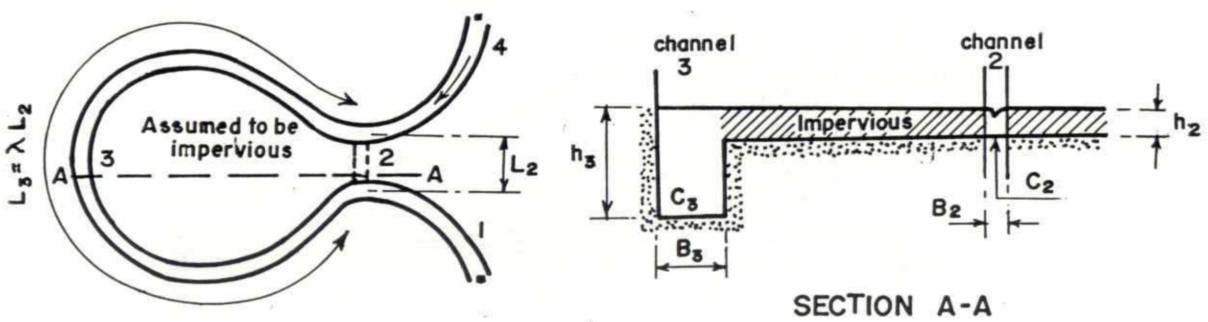

Fig. 5-17: Definition of cut-off ratio

For a braided sand bed river like the Jamuna River, a different criteria is 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:

$$\frac{S_{2\ incoming}}{S_{3\ incoming}} = \frac{1}{v}\frac{Q_2}{Q_3} \tag{5.3}$$

where v = parameter which determines the distribution of the sediment at the potential bifurcation. The resulting design graph is shown in Fig. 5-18.

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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 2 for recent studies on the importance of this geometry).

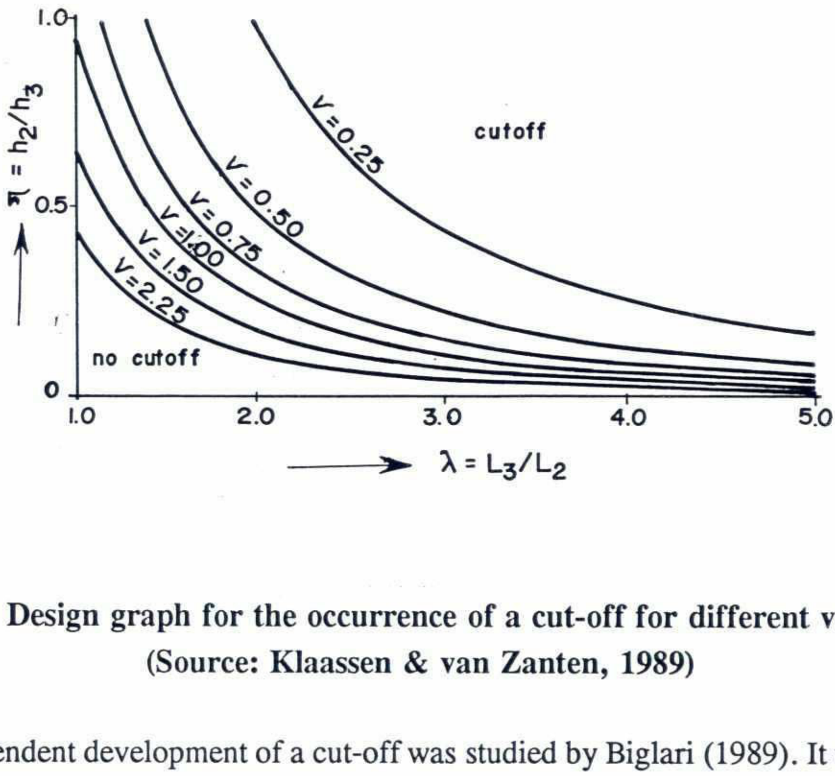

Fig. 5-18: Design graph for the occurrence of a cut-off for different values of *v* (Source: Klaassen & van Zanten, 1989)

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):

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

For definitions see Fig. 5-17 . It follows that a minimum value for T is found if $S_{2\ incoming} = 0$, but the time required can take much longer if $S_{2\ incoming}$ has a value. For $S_{2\ incoming} = S_{transport\ capacity}$ 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-off 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. 5-18 a cut-off for λ = 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 Chapter 3 of Annex 2). 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 upto 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 5-2.

Recurrent measure	At bifurcation	In outer channel	At outer bank
dredging	-	-	-
revetment	-	-	recurrent structures
dikes	permeable groyne	sill	permeable groyne
vanes	surface vanes	bottom vanes surface vanes	-
jacks	-	-	-
artificial cut-off	To be considered later		

Table 5-2: Preliminary selection of recurrent measures

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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 line 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 a row of vanes along the bank is preliminarily selected.
- 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 upto 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 (verbal 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

PART C

MORPHOLOGICAL RESPONSE TO MEASURES AND STRATEGIES

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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 (the hydraulic 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 is seeking the relation between the study of the measures (e.g. main dimensions) and the erosion process. This is done in steps as indicated in Table 6-1, and in Fig. 6-1.

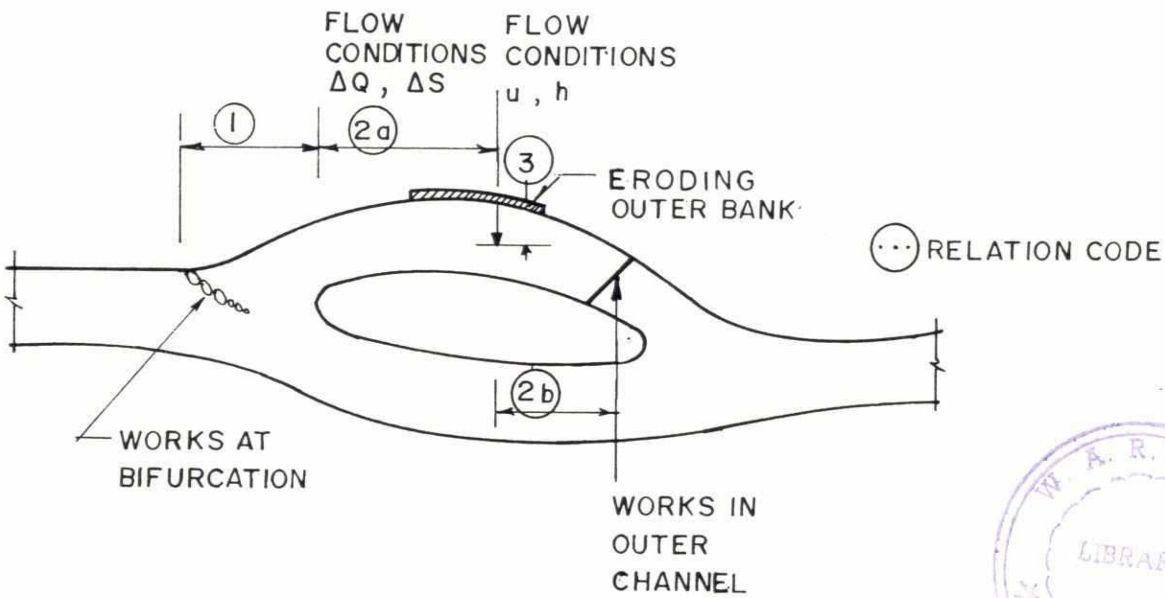

Fig. 6-1: Measures-erosion relation

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. low 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 verbal 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 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. 6-2).

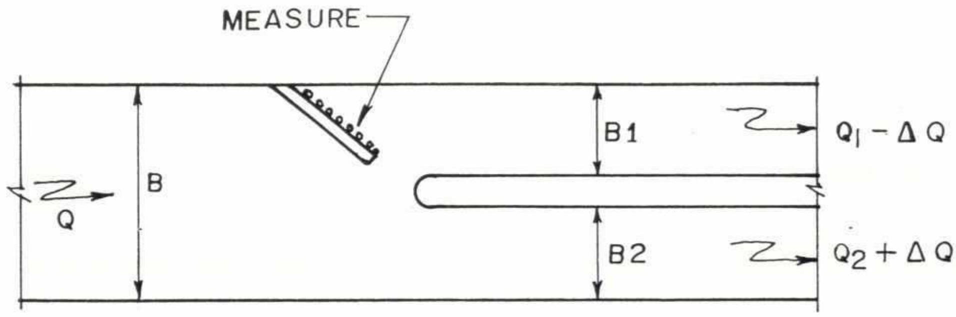

Fig. 6-2: Schematization of bifurcation

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 slope. The velocities in the channels may be different.
10. For the situation with the bandal, the flow field need 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 bandals 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.

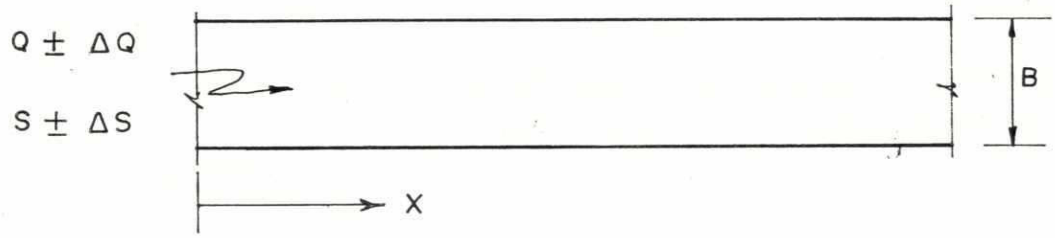

Fig. 6-3: Response per channel

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. 6-4.

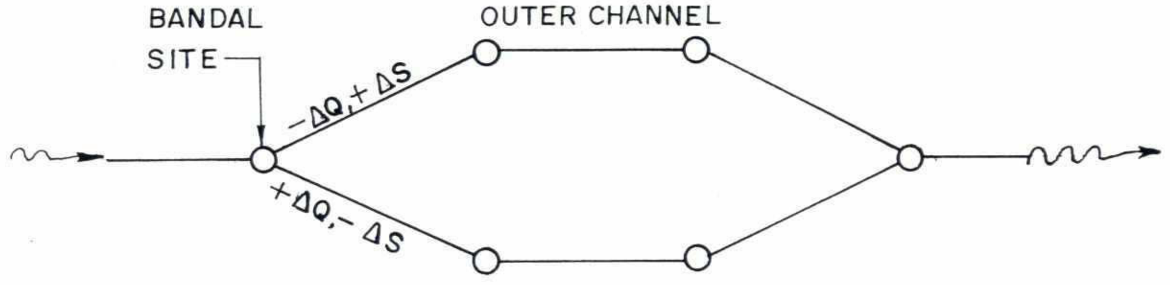

Fig. 6-4: System schematization

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. The objective of the computation is to find u,h = f(h_s/h_o, t, ...). Some characteristic details have been presented in Fig. 6-5.

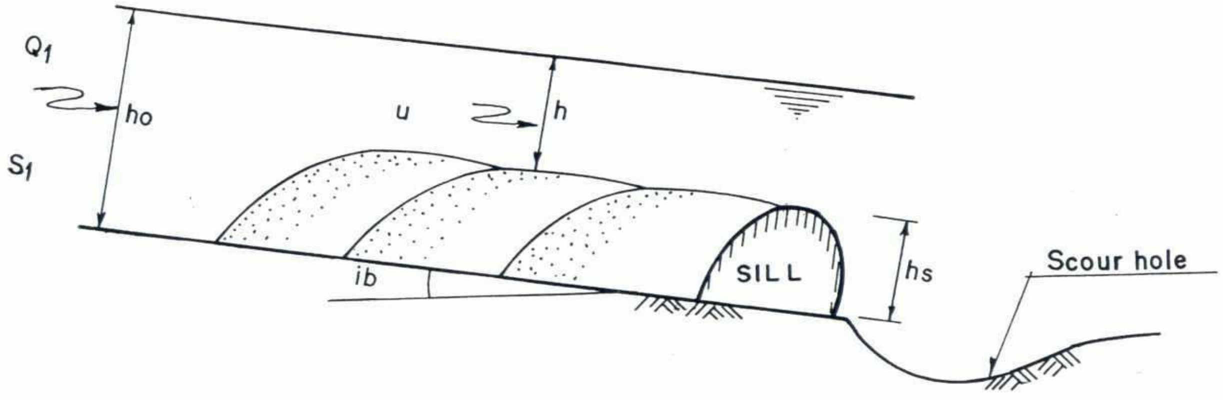

Fig. 6-5: Sedimentation upstream sill

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.

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.

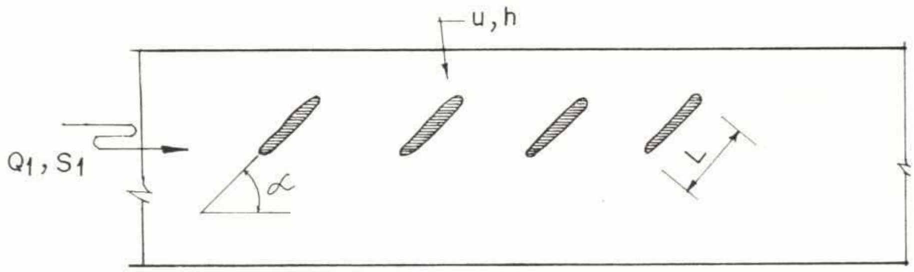

Fig. 6-6: Bottom panels in local model

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.

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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.

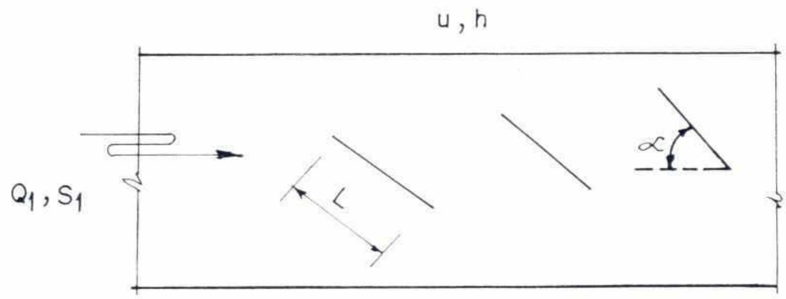

Fig. 6-7: Surface vanes in local model

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 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

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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 vertical bank angle slope

This contribution to the bank retreat vanishes for vertical banks.

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 substantially, 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:

$$\frac{\delta n_B}{\delta t} = E\,\frac{(u_w^2 - u_{wcr}^2)}{u_{wcr}^2} + G.\frac{h_w - h_{wcr}}{h_{wcr} + H_{fb}} - \frac{1}{\tan\varphi}.\frac{\delta z_b}{\delta t} \tag{6.1}$$

in which $\delta n_B / \delta t$ = the bank retreat as a function of time (m/s), E = erodibility coefficient, G = erodibility coefficient, 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:

$$\Omega = \rho g Q i \tag{6.2}$$

with

ρ = density of water		(kg/m²)
g = acceleration of gravity		(m/s²)
Q = river discharge		(m³/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. 6-8.

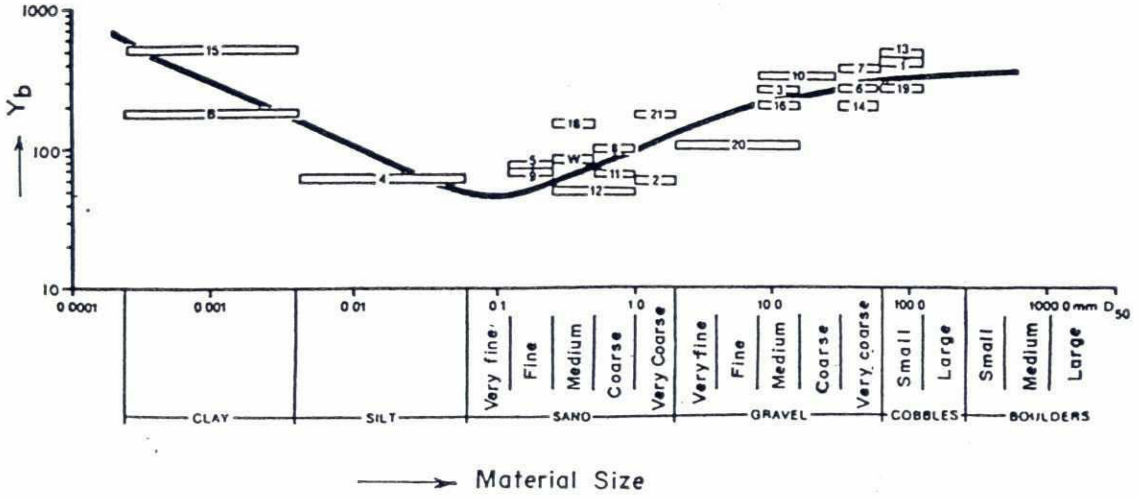


Fig. 6-8: Relation between coefficient of resistance to lateral erosion and bank material size
(Source: Hickin & Nanson, 1984)

Hickin & Nanson (1984) demonstrate on the basis of a number of rivers (as indicated by individual figures in the graph presented in Fig. 6-8) in Canada that the bank erosion rate can be computed with the following relations:

$$E = \frac{2.5}{(R/B)} M_{2.5} \quad (for\ \frac{R}{B} > 2.5)$$
$$E = \frac{(R/B-1)}{1.5} M_{2.5} \quad (for\ \frac{R}{B} < 2.5) \qquad (6.3)$$

where E = annual erosion (m), $M_{2.5}$ = annual erosion for R/B = 2.5 (m), 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:

$$M_{2.5} = \frac{\Omega}{Y_b H} \qquad (6.4)$$

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. 6-9 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) analyzed data from the Jamuna River and found a similar shape of the bank erosion rates (see Fig. 6-9 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.

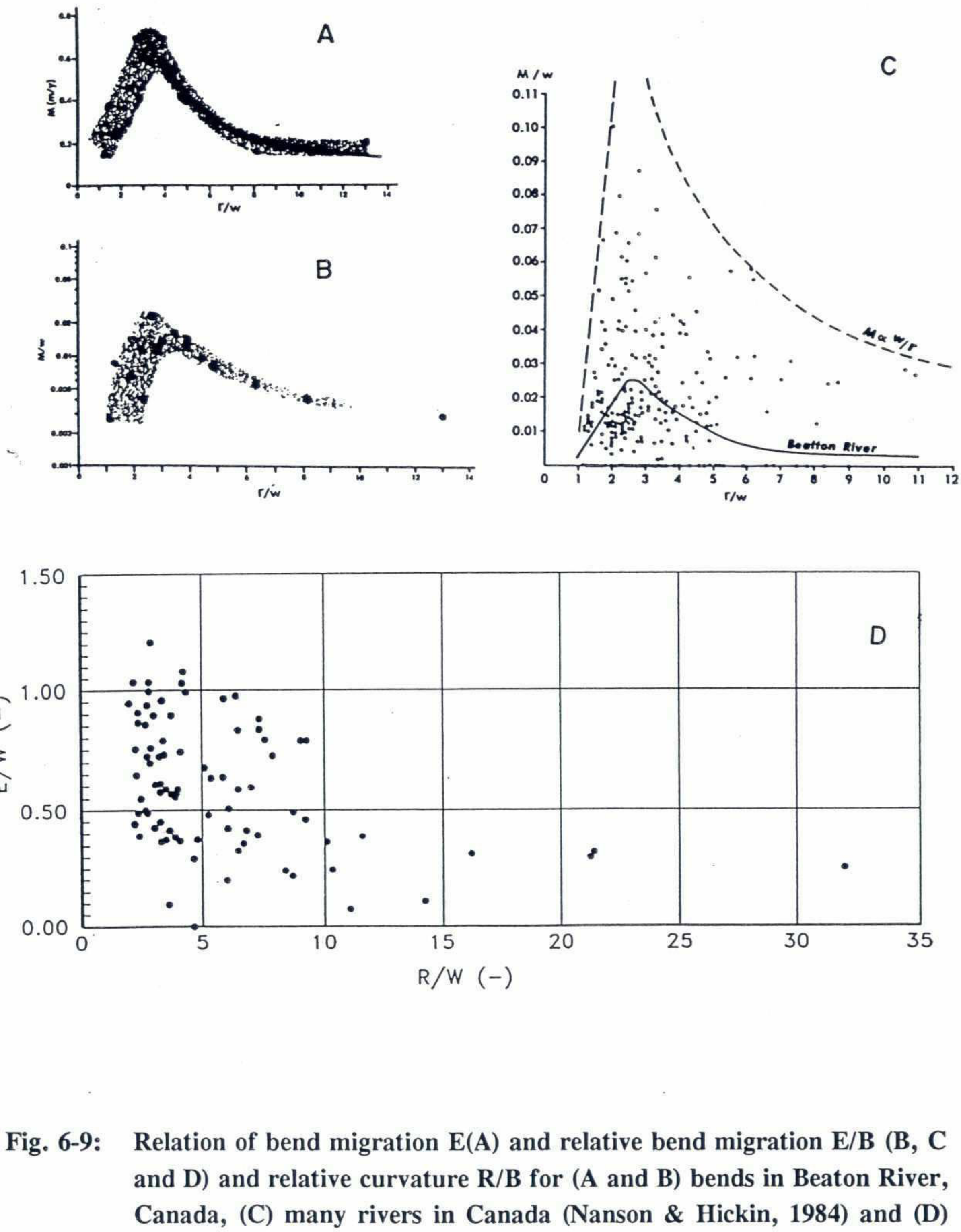
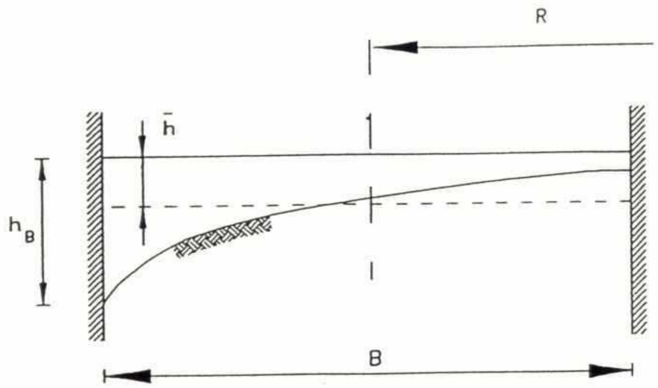


Fig. 6-9: Relation of bend migration E(A) and relative bend migration E/B (B, C and D) and relative curvature R/B for (A and B) bends in Beaton River, Canada, (C) many rivers in Canada (Nanson & Hickin, 1984) and (D) Jamuna River, Bangladesh (Klaassen & Masselink, 1992)

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

$$H = h_b + \Delta h_B - \Delta h_{erosion\ products} \tag{6.5}$$

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. 6-10. It appears that Δh_B is a function of h_b, 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.



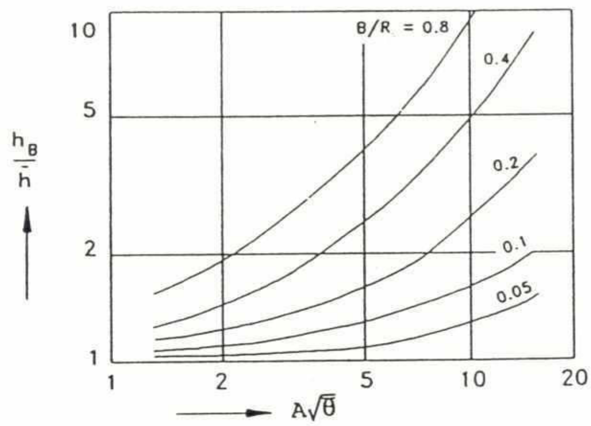


Fig. 6-10: Maximum bend scour
(Source: Mesbahi, 1992)

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

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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 reduce 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 Technical Report. 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 discharge that the floodplain starts to inundate (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 (see again 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 (we will come back on this later), the following parameters can be identified in a braided river system: the dominant discharge Q_b, the sediment transport the river has to carry S, 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 of the river i, 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 will be shown later in a separate Technical 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_b, 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. 7-1, 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.

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 depend 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.

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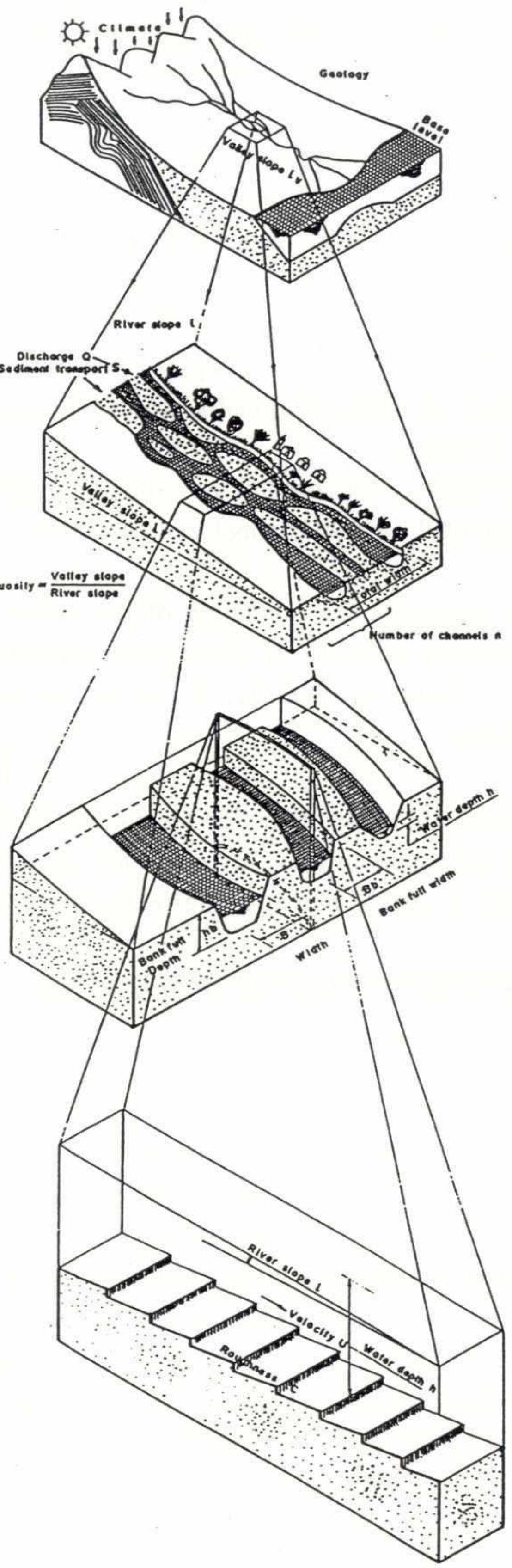

Fig. 7-1: Main parameters in a river system

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. It may be assumed that narrowing of the river is not a feasible option.

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 (5). 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:

$$P = 2.67\, Q^{\frac{1}{2}} \tag{7.1}$$

$$R_h = 0.473\, Q^{\frac{1}{3}} f^{-\frac{1}{3}} \tag{7.2}$$

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

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:

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$$f = 1.76 \sqrt{D_{50}} \tag{7.4}$$

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:

$$B = C_1 \, Q^{0.5} \tag{7.5}$$

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:

$$i = 0.059\ Q_2^{-0.333}\ D_{50}^{0.586} \tag{7.6}$$

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

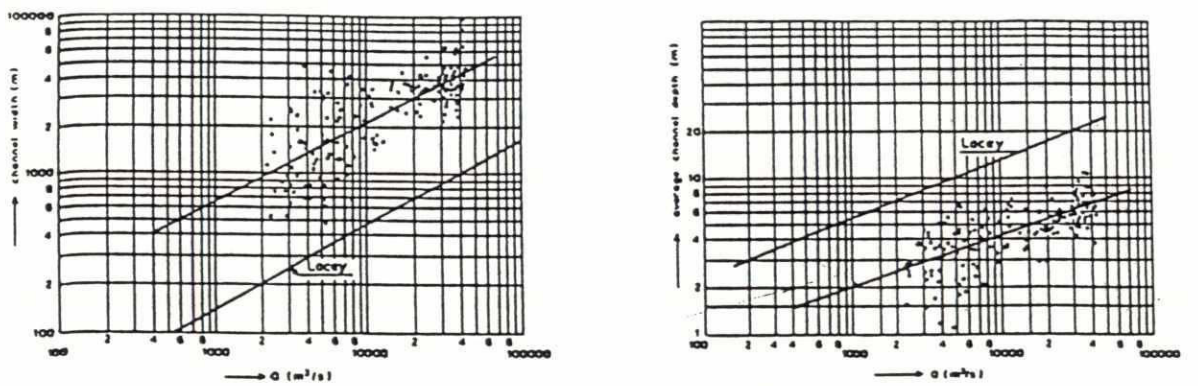
Relations for braided sand bed rivers are given by Klaassen & Vermeer (1988). In fact these equations were derived from an analysis of cross-sections from the Jamuna River, and some spurious correlation was introduced, because the bankfull discharge was divided over the channels according to their conveyance. 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:

$$\overline{h_b} = 0.23\ Q^{0.32} \tag{7.7}$$

$$B_b = 16.1\ Q^{0.53} \tag{7.8}$$

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

This is also shown in Fig. 7-2, 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. 7-2: Regime equations for Jamuna River channels compared with the Lacey (1929) regime equations (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 shape that satisfy stable conditions. Related approaches have been followed by Thorne (1982), which related the channel stability to the stability of the banks, and Singh (1983), related 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:

$$\frac{h}{D} = 85.1 \left[\frac{w_s}{\left(\frac{\rho_s}{\rho} - 1\right) g D^{1/2}}\right] i^{-1/2} \qquad (7.9)$$

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 compatable 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 paramenter, 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, 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.

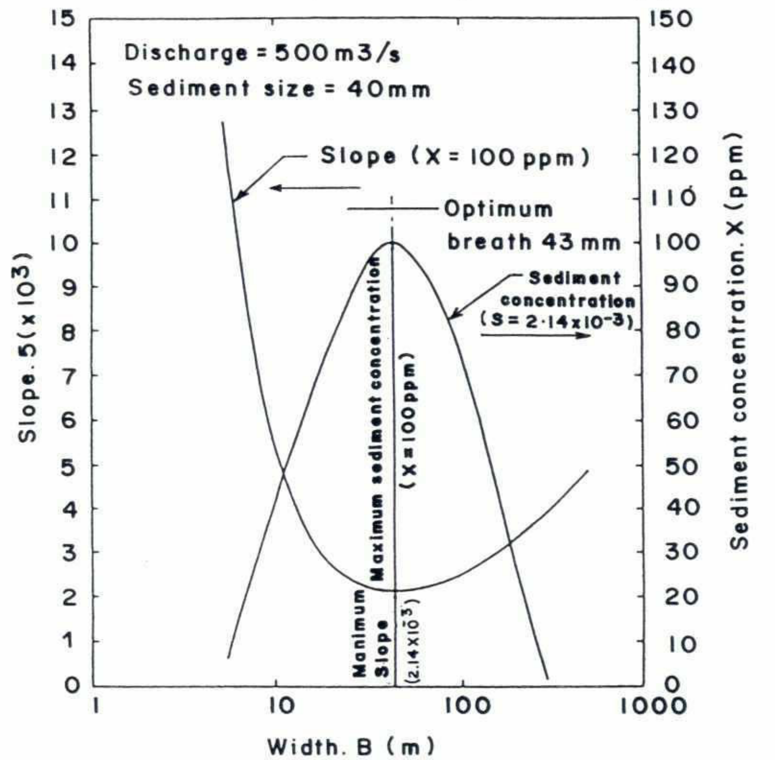


Fig. 7-3: Example of variation of slope and sediment concentration as a function of width (White et al, 1982)

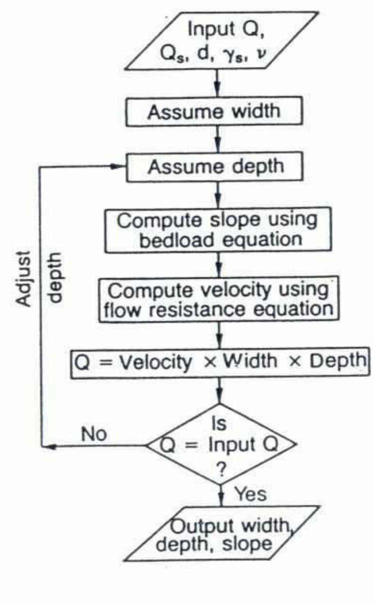


Fig. 7-4: Flow chart showing major steps in computation of Chang (1979)

In the same Fig. 7-3 the result of an analysis keeping the sediment transport constant (to 100 ppm) and again varying the width are presented. In this case, corresponding to the minimum power hypothesis of Chang (1979), 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. 7-4 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 7-1 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:

$$z = 0.5 \qquad if \quad Q < 1\ m^3 / s \tag{7.10}$$

$$z = 0.5\ Q^{\frac{1}{4}} \qquad if \quad Q > 1\ m^3 / s \tag{7.11}$$

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		Prediction method					
Predictor	Equation	Chang (1979; 1980 and more recent)			Yang (1979 and more recent)	Ramette (1979; 1990)	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 7-1: Comparison of different extremal hypotheses proposed

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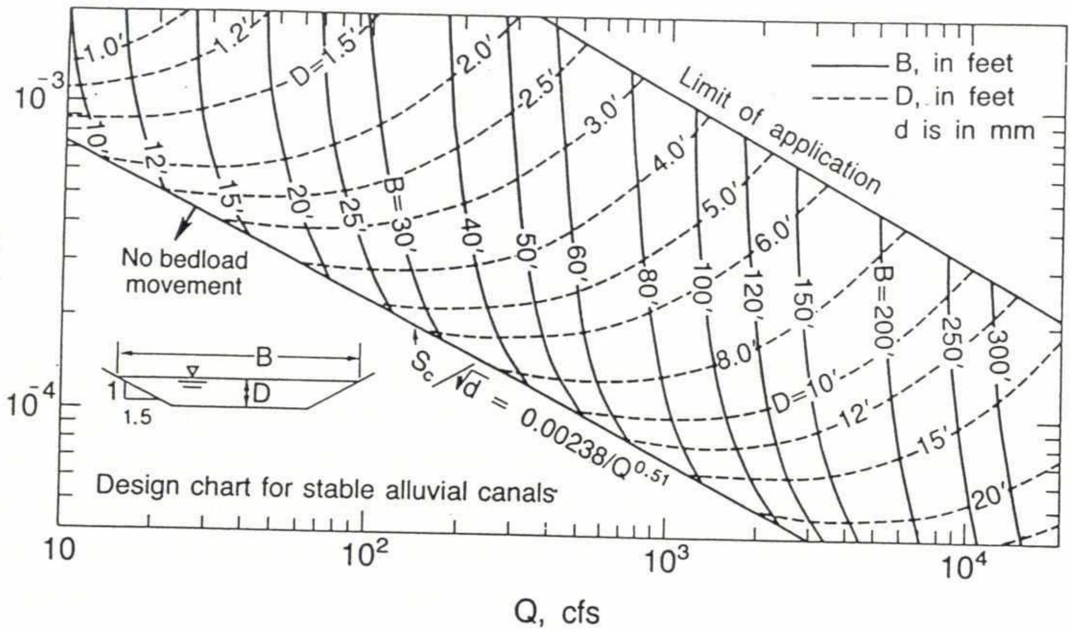
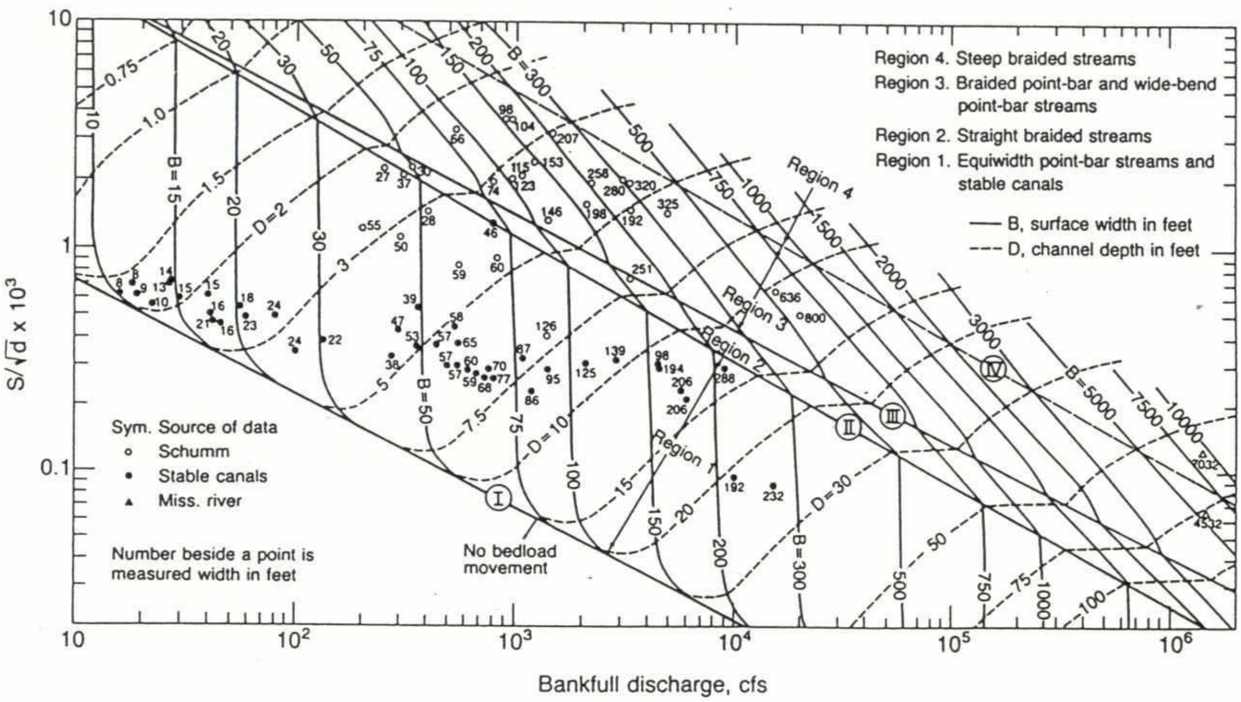


(a) Stable channels



(b) Sand-bed channels

Fig. 7-5: Design graphs developed by Chang (1979, 1980)

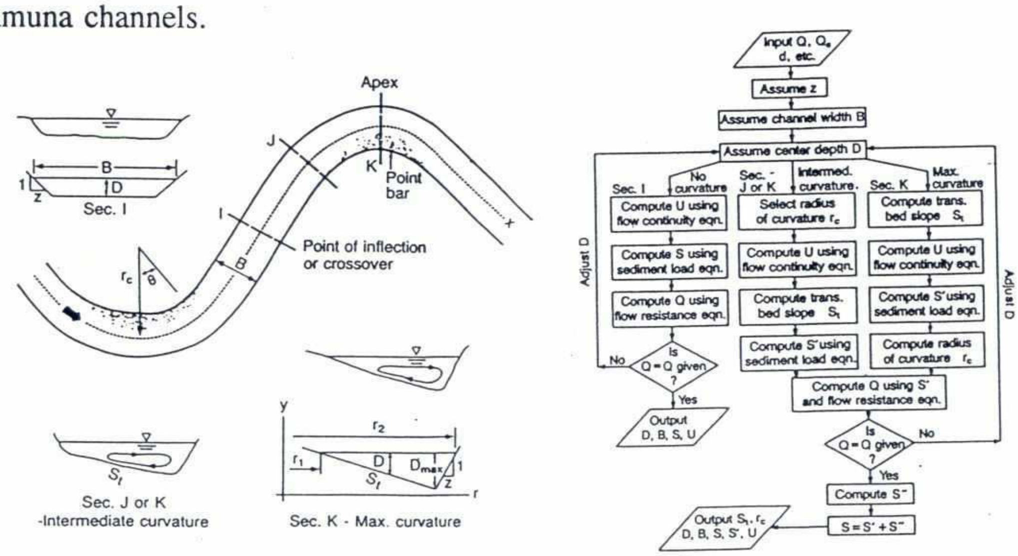
Different researchers have elaborated the use of the extremal hypothesis up to different levels:

(1) Chang (1987) has developed design charts for stable alluvial channels which are dependent on the accepted side slope (see Fig. 7-5a). 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 the effect of meanders, arriving at design graphs for sand bed rivers. As is shown in Fig. 7-6 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. 7-5b. 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. 7-7, 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. 7-6: Extremal hypothesis by Chang (1980) applied to meandering sand-bed rivers

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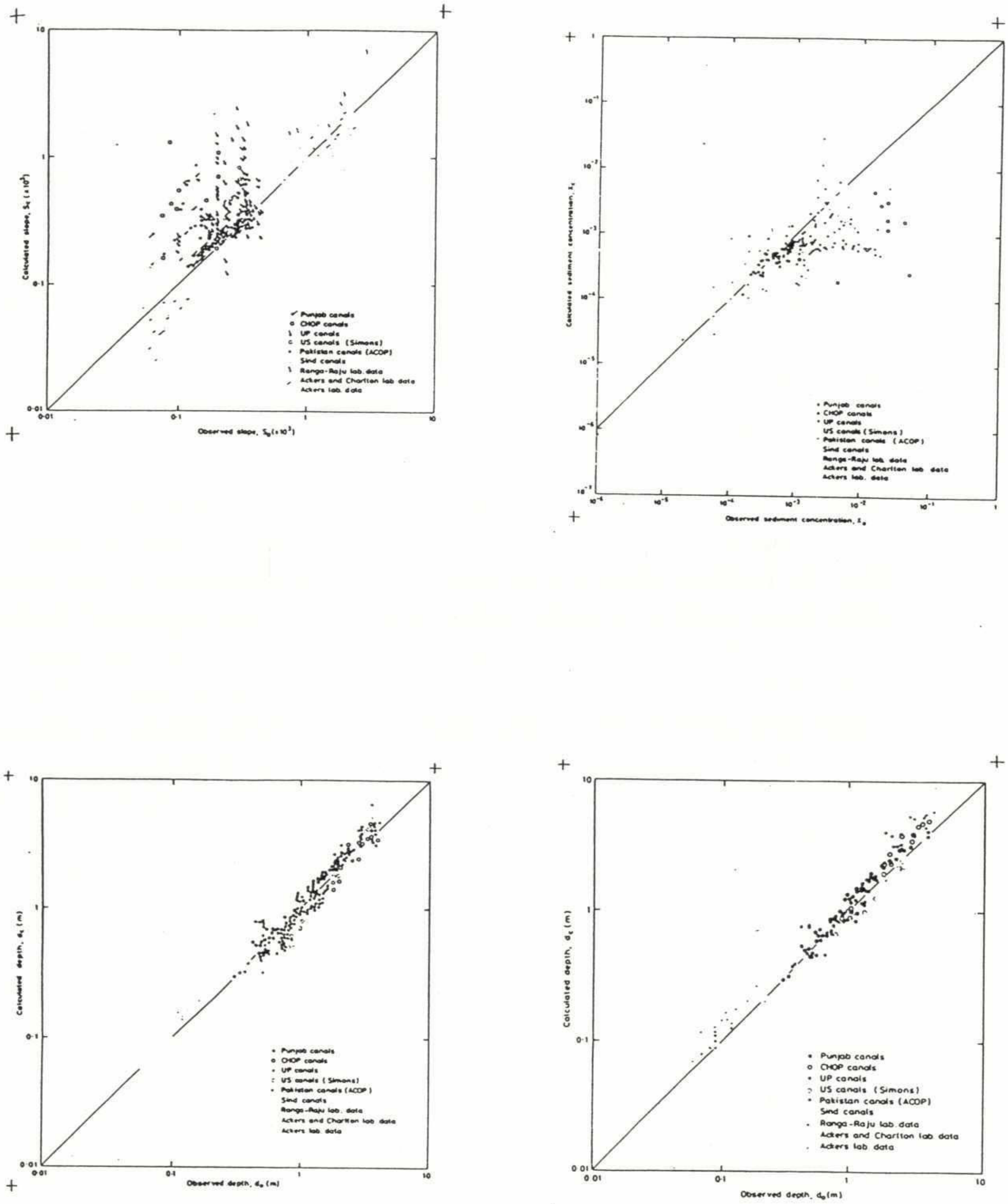

Fig. 7-7: Comparison of method of maximum sediment transport method of White et al with (mostly) field data (from White et al, 1982)

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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:

$$R = 9.5 * 10^{-2}\ Q^{0.233} D^{0.26} i^{-0.41} \tag{7.12}$$

$$B = 8.4\ Q^{0.527} D^{0.006} i^{-0.077} \tag{7.13}$$

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 (7.7) and (7.8) yields:

$$R = 0.54\ Q^{0.233} \tag{7.14}$$

$$B = 16.9\ Q^{0.527} \tag{7.15}$$

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 convential 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:

$$p = \frac{i_v}{i} \tag{7.16}$$

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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 and 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. 7-8.

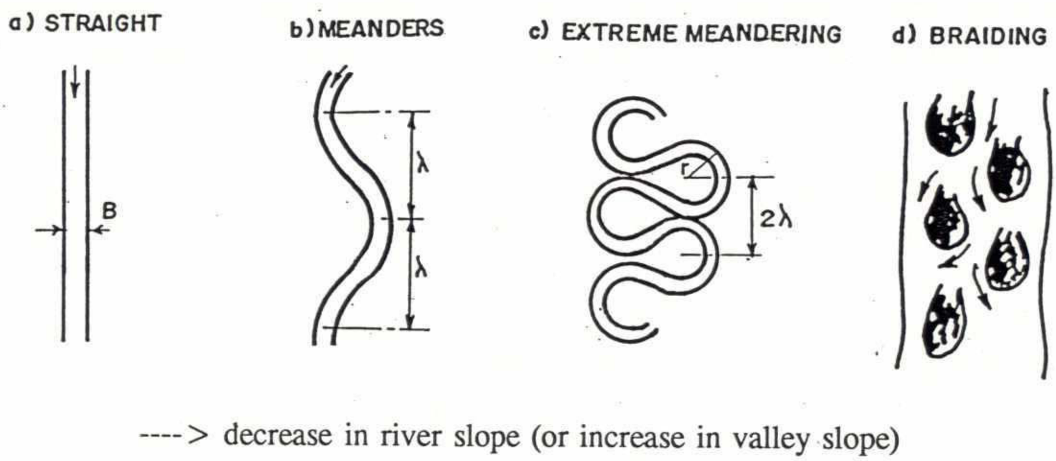

Fig. 7-8: Meandering as a way to cope with a difference between valley slope and river slope
(Source: Ramette, 1990)

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 form one channel to more channels (usually associated with the transition form 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

$$i = 0.0116\ Q^{-0.44} \tag{7.17}$$

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.

$$\lambda_w = \frac{C^2}{2g} h_0 \tag{7.18}$$

for water motion, and

$$\lambda_s = \frac{1}{m^2 \pi^2} \frac{B^2}{h_0^2} f(\theta_0) h_0 \tag{7.19}$$

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

$$\lambda_s(m) = \frac{1}{m^2} \lambda_s(1) \tag{7.20}$$

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), 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. 7.9 illustrates the response of a river to increasing valley slopes according to Bettess & White (1983).

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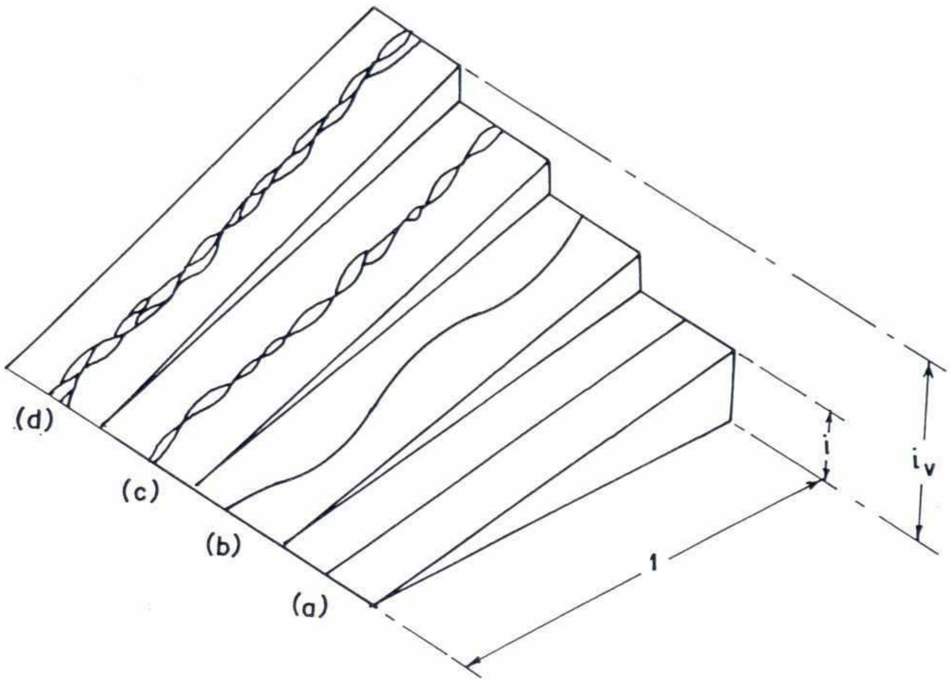

Fig. 7.9: Channel pattern for increasing valley slope (Source: Bettess & White, 1983)

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.

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 frame-work 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

$$\frac{m^2}{2}\frac{\lambda_w}{\lambda_\xi(1)}\frac{1}{(k\lambda_w)^2}+\xi=\frac{(2+X)(1+bX)+(1+X)(b-3-bX)}{4(k\lambda_w)^2(1+X)^2+(2+X)^2} \tag{7.21}$$

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

$$X=\frac{kB}{m\pi} \tag{7.22}$$

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.

$$\frac{\lambda(1)}{\lambda_w}=F_m\left(\frac{kB}{\pi}\right) \tag{7.23}$$

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

$$m_{\bullet} = f(m) \tag{7.24}$$

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 $m_{\bullet}$ and m. 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:

$$A = C_1\, Q^{C_2} \tag{7.25}$$

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 what is known as far as prediction methods for channel characteristics of braided river systems with fine sand as bed and bank material is concerned. In the 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 extremal 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 Verification

Considering the above an important step in the selection procedure is the verification of the applicability of the various proposed methods against data on the Jamuna River. What has to be done in the near future is that the river characteristics of the Jamuna River are collected and that in a further step the different prediction methods are compared with these river data. During this comparison both theoretical and empirical predictors should be verified. The preference is of course for theoretical predictors, but in the experience of Consultants even theoretical predictors often have to be calibrated using field data. If theoretical predictors are not applicable, it will be necessary to use empirical relationships. If necessary, even empirical methods especially developed for the Jamuna River have to be developed.

7.4.3 Prospects

The main question to be answered at the end of this Chapter is whether there are good prospects for the prediction of the response of the Jamuna River system to FAP 22 activities. Based on the review given above it can be stated that some theoretical understanding is present that can be used as a basis. For the time being however it seems that one has to rely mainly on empirical methods. An example of a preliminary application of empirical prediction methods to assess the response of the river system to FAP 22 measures is presented in Section 3.6 of the Interim Report. It may be assumed that based on the verification phase which is now ongoing an improvement of the methods used there can be achieved, allowing for a slightly better prediction.

Finally it should be remarked that all methods discussed in this Chapter are dealing with ultimate conditions. Intermediate stages have to considered as well, using e.g. the techniques described in Chapter 6.

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References

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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