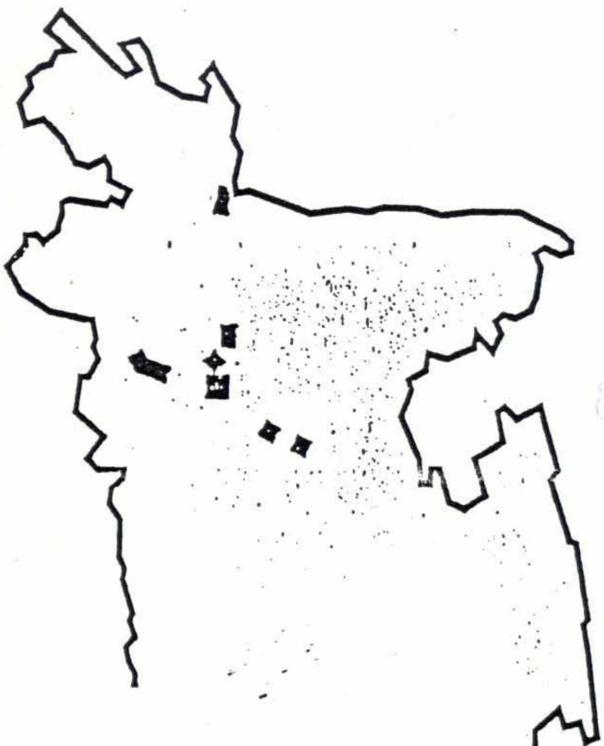
GOVERNMENT OF BANGLADESH
FLOOD PLAN COORDINATION ORGANIZATION

FAP 24 RIVER SURVEY PROJECT

RIVER DYNAMICS



DELFT HYDRAULICS
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RIVER DYNAMICS

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Engineering Potamology
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INTERNATIONAL INSTITUTE FOR HYDRAULIC AND ENVIRONMENTAL ENGINEERING

E N G I N E E R I N G   P O T A M O L O G Y

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Contents

1. Introduction

1.1. General

Percipitation (rain or snow) leads to run-off. It also leads to soil erosion. Water and sediment are transported down-hill to the sea or ocean. This transport takes place via rivers of various sizes and shapes. Knowledge of the natural processes in rivers (*river hydraulics or potamology*) is essential to understand and predict changes that will occur due to natural causes or due to human interference by *river engineering works*.

The combined transport of water and sediment is a three dimensional time depending phenomenon, which is of a complex nature. A complete *deterministic* description fails due to the *stochastic character* of the morphological processes. At best a rather schematic approach can be used starting from the equations of motion and continuity of two phases: water and sediment. This, however, is only possible when an *alluvial channel* is involved i.e. the river flowing to its own non-cohesive sediment. The picture can be completely different if the natural river differs from this idealized case. This is for instance the case when the alluvial bed contains resistant spots (clay or rock).

Another example that makes morphological prediction for rivers extremely difficult is the occurence of extremely rare high discharges that cannot be predicted. Their influence on the fluvial processes, however, can be extremely large. This is due to the strongly non-linear relationship between water movement and sediment movement.

- El Niño, a yearly dislocation in one of the world's largest weather systems over the Pacific Ocean had a large global impact in 1982-1983.
  It has enormous consequences in terms of floods and droughts (Canby, 1984). Among other things the usual flood of the Chira River (Peru) in the beginning of the year became so extremely large early 1983, that the river changed its downstream course over many kilometers due to this single flood.
  Early 1984 the tropical cyclone Demoina stayed long time over the Southern part of Mozambique. De rivers Maputo, Umbeluzi and Incomati obtained extremely large discharges as locally 700 mm of rainfall occured in a few days. Estimated discharges were about ten times higher than the recorded maximums.

These examples should be kept in mind when morphological forecasts have to be made: the predictions can be based on the statistical properties of the discharge. However, one extreme non-predictable flood can change the whole situation.

The characteristics of rivers can vary largely due to the properties of the rainfall, the characteristics of the catchment area (elevation, soil properties, vegetation, etc.) and the influence of men in the river system.

These aspects will only be treated briefly in the following Sections of Chapter 1. In Chapter 2 some essential river characteristics are treated, whereas Chapter 3 is dealing with fluvial processes due to the combined transport of water and sediment. Finally the principle of morphological predictions is discussed in Chapter 4. These predictions are necessary to forecast the morphological changes due to river engineering works.

1.2. Hydrological aspects

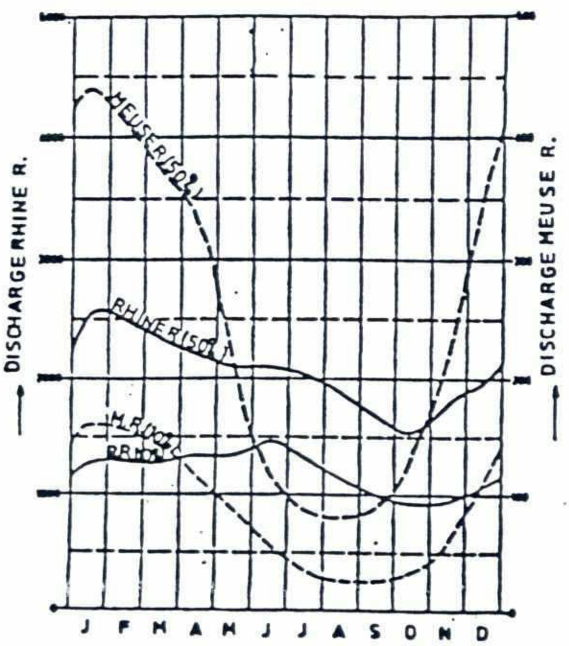

Fig. 1.1. Discharge Rivers Rhine and Meuse

Many aspects govern the shape of the discharge curve Q(t) of a river. In Fig. 1.1 one aspect has been demonstrated. For the River Rhine a substantial part of the precipitation is in the form of snow. The run-off is then retarded. Part of the discharge comes from the glaciers in Switserland. The course of Q(t) is therefore rather regular. This is contrary to the River Meuse, directly fed by rain on the catchment area which has little storage. Another example of a river with relatively small differences between the yearly maximum and minimum discharges is formed by the Congo River near Brazzaville (Fig. 1.2). The values for 1983 were at Brazzaville Q_{max} = 77 400 m^3/s and Q_{min} = 23 000 m^3/s. Thus Q_{max}/Q_{min} = 3.5.

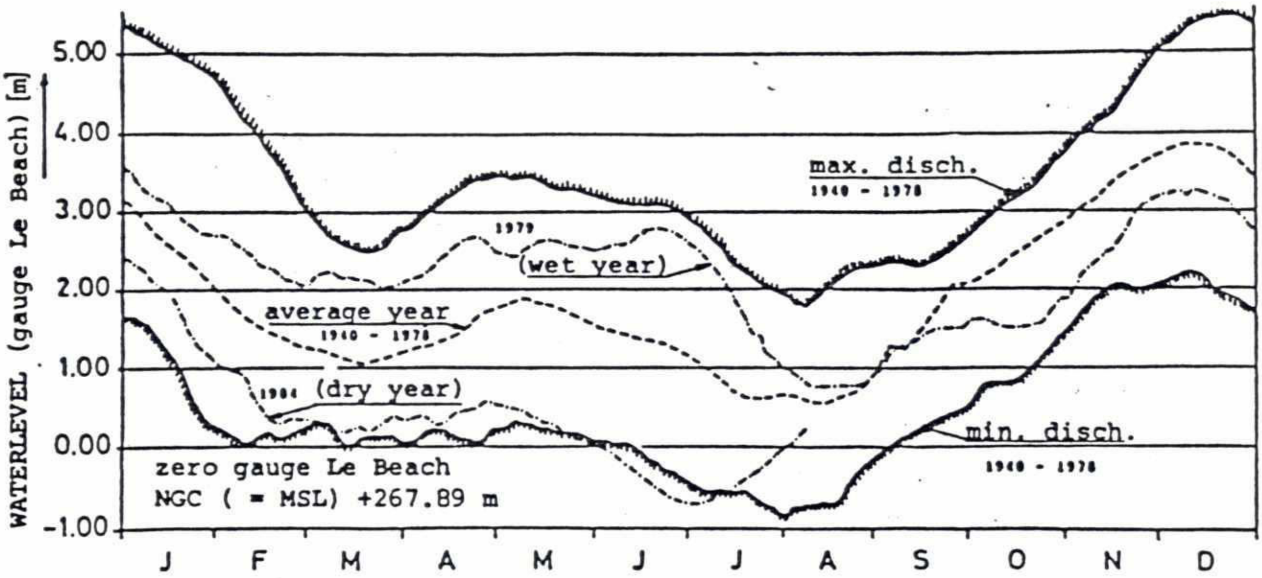

Fig. 1.2. Congo River near Brazzaville

The above given examples regard *perennial rivers*: there is a substantial discharge throughout the year.

On the other hand there are *ephemeral rivers*: during a large period of the year there is little or no discharge. Discharge takes place during a short period in the rainy season. An example is the Choshui River on Taiwan Island. (Fig. 1.3).

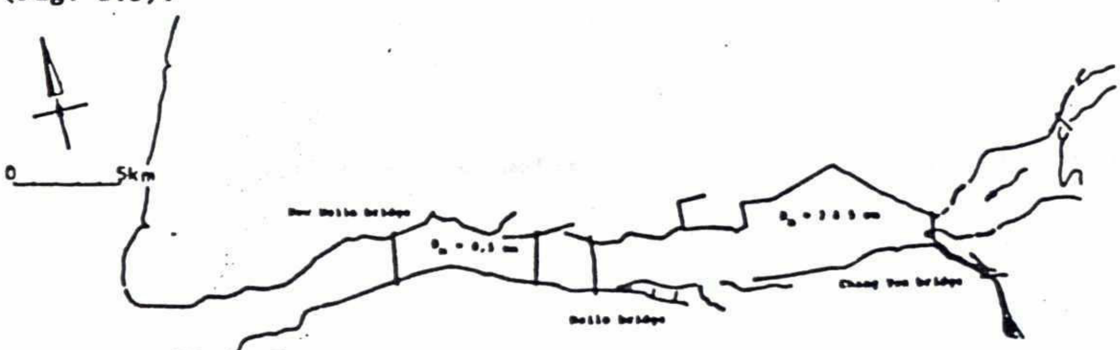

Fig. 1.3. Lower part of Choshui River (Taiwan)

During the typhoon period (July-September) this river carries a substantial discharge. During the rest of the year the river bed is almost dry. There is a substantial rainfall in the catchment area (2 555 mm/a) but the rain is concentrated. Therefore in spite of the fact that the catchment area is relatively small (3 155 km^2) substantial discharges can occur. For the River Choshui the once-in-hundred years discharge in Hsi-Lo amounts to 24 000 m^3/s.

The annual hydrograph Q(t) (or, if expressed in water levels h(t)) is partly due to the pattern of the rainfall, R(t). The regular shape h(t) for the River Congo near Brazzaville (Fig. 1.2) is partly due to the fact that 2/3 of the catchment area is located on the Southern Hemisphere while 1/3 is situated on the Northern Hemisphere. Therefore yearly two monsoons are present. This leads to two low-water periods. For the River Congo two other aspects play a role. The catchment area is not very ondulated and is heavily vegetated. Both aspects contribute to the regular pattern of h(t).

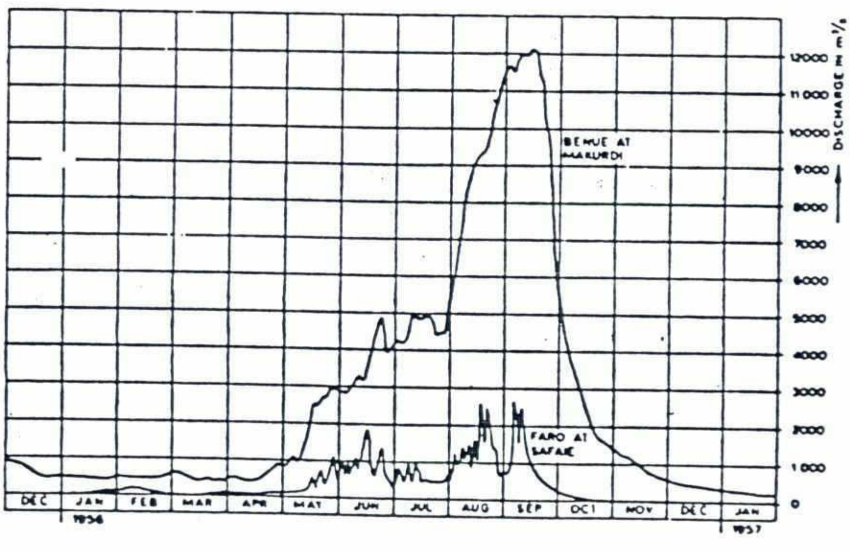

Fig. 1.4. Discharges for River Benue and River Faro.

In Fig. 1.4 the hydrograph Q(t) of the River Benue is given. This is the main tributary of the River Niger. In this figure also the hydrograph of the River Faro, a small tributary of the River Benue is given. The River Faro with its small catchment area is much more flushy than the much larger River Benue. Discharge peaks of tributaries are demped in the main river.

1.3. Geological aspects

The geology of the river basin is an important factor in the appearance. The discharge and sediment transport are characterized by the the catchment area (variation in elevation, erodibility, vegetation, etc.). Consequently a large variation in rivers is present. Table 1.1 gives some general information.

			Discharge				
		Catchment area	Water		Sediment		Sediment as ppm of discharge ($mg\ l^{-1}$)
River	Station	$10^6\ km^2$	$m^3\ s^{-1}$	$mm\ yr.^{-1}$	10^6 ton $yr.^{-1}$	$10^{-3}\ mm\ yr.^{-1}$	
Amazon	mouth	7.0	100 000	450	900	90	290
Mississippi	mouth	3.9	18 000	150	300	55	530
Congo	mouth	3.7	44 000	370	70	15	50
La Plata/Parana	mouth	3.0	19 000	200	90	20	150
Ob	mouth	3.0	12 000	130	16	4	40
Nile	delta	2.9	3 000	30	80	15	630
Yenissei	mouth	2.6	17 000	210	11	3	20
Lena	mouth	2.4	16 000	210	12	4	25
Amur	mouth	2.1	11 000	160	52	15	150
Yangtse	mouth	1.8	22 000	390	500	200	1 400
Wolga	mouth	1.5	8 400	180	25	10	100
Missouri	mouth	1.4	2 000	50	200	100	3 200
Zambesi	mouth	1.3	16 000	390	100	50	200
St Lawrence	mouth	1.3	14 000	340	3	2	7
Niger	mouth	1.1	5 700	160	40	25	220
Murray-Darling	mouth	1.1	400	10	30	20	2 500
Ganges	delta	1.0	14 000	440	1 500	1 000	3 600
Indus	mouth	0.96	6 400	210	400	300	2 000
Orinoco	mouth	0.95	25 000	830	90	65	110
Orange River	mouth	0.83	2 900	110	150	130	1 600
Danube	mouth	0.82	6 400	250	67	60	330
Mekong	mouth	0.80	15 000	590	80	70	170
Hwang Ho	mouth	0.77	4 000	160	1 900	1 750	15 000
Brahmaputra	Bahadurabad	0.64	19 000	940	730	800	1 200
Dnjepr	mouth	0.46	1 600	110	1.2	2	25
Irrawaddi	mouth	0.41	13 000	1 000	300	500	750
Rhine	delta	0.36	2 200	190	0.72	1	10
Magdalena (Colombia)	Calamar	0.28	7 000	790	220	550	1 000
Vistula (Poland)	mouth	0.19	1 000	160	1.5	5	50
Kura (USSR)	mouth	0.18	580	100	37	150	2 000
Chao Phya (Thailand)	mouth	0.16	960	190	11	50	350
Oder (Germany/Poland)	mouth	0.11	530	150	0.13	.1	10
Rhone (France)	mouth	0.096	1 700	560	10	75	200
Po (Italy)	mouth	0.070	1 500	670	15	150	300
Tiber (Italy)	mouth	0.016	230	450	6	270	850
Ishikari (Japan)	mouth	0.013	420	1 000	1.8	100	140
Tone (Japan)	Matsudo	0.012	480	1 250	3	180	200
Waipapa (New-Zealand)	Kanakanala	0.0016	46	900	11	5 000	7 500

Table 1.1. Some basic data of rivers (after Jansen, 1979)

More information on sediment production in river basins is provided by Fournier (1969). In Table 1.1 the rivers are listed by the length of the main stem.

The difference in average sediment concentration is large. The champion is the Yellow River (Huang He) in China. This river is flowing through a loess-area leading to a substantial transport of fine material.

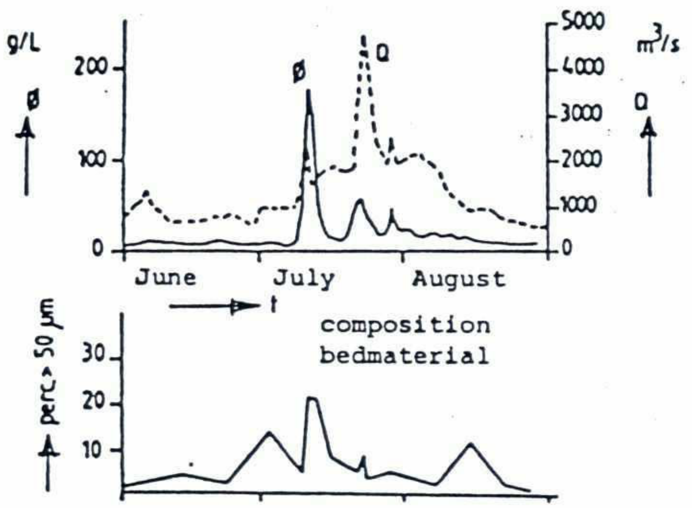

Fig. 1.5. Example sediment transport Yellow River (Long & Xiong, 1981)

Figure 1.5 shows some transport measurements at the station Tungkuan. Concentrations upto 175 g/liter do occur.

The Yehe River, a tributary upstream of this station flows through a hilly loess-area with quite some *gully erosion*. There the mean concentration is even more than 300 g/liter. The Yehe River has a catchment area of only 3208 km^2; the sediment yield is above 14400 t/km^2.a. On the other hand the St. Lawrence River (Canada) is carrying very little sediment; this river flows through a number of lakes.

The present geological processes can still influence a river basin. Near the confluence of the River Magdalena and the River Cauca (Columbia) the Island Mompos is situated. This area is due to subsidence caused by the tectonics in the Andes. Under natural conditions the subsidence is balanced by the yearly sedimentation during floods.

Another example is reported by Murty (1973). Due to earthquakes in the Himalayas slidings occur which bring suddenly and locally large amounts of sediment in the Brahmaputra River. This causes the low water levels and the high water levels to rise (Fig. 1.6).

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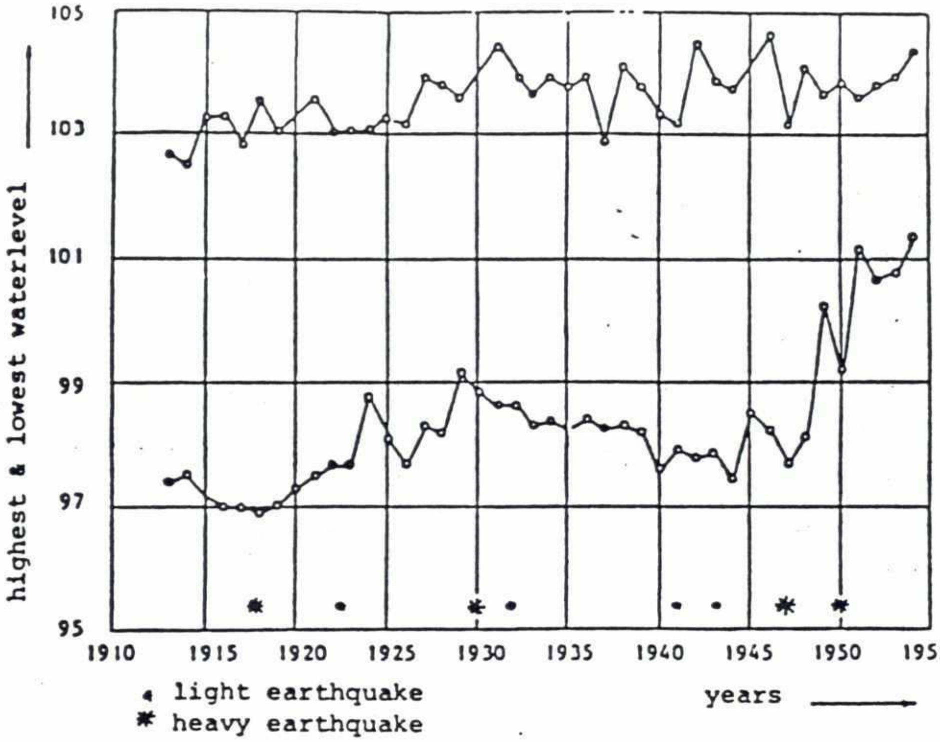

Fig. 1.6. Water level rises Brahmaputra River indirectly due to earthquakes (Murty, 1973)

The composition of the rock that is the source of the sediments (= erosion products) determines the morphological processes.

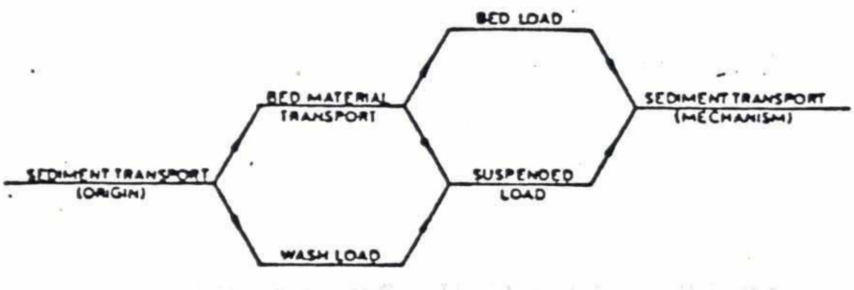

Fig. 1.7. Classification of transport.

In Fig. 1.7 the usual (qualitative!) definitions of the various modes of transport are given:

Bed material transport is the transport of the size fractions that are present in the bed material of the river.

Washload is the transport of the fine particles that are not found in appreciable quantities in the bed.

The bed material transport is determined by the composition of the bed and by the hydraulic characteristics of the stream. It can be determined by transport formulae.
On the other hand there is washload. The amount of washload in a reach is only determined by the upstream supply. Hence it is not determined by the hydraulic parameters of the stream.
This brings forward a problem because a sediment-water sample taken from the stream will contain sediment belonging to the bed material load as well as to the washload (Fig. 1.7).

In practice depending of the geological features of the catchment area rivers can be distinguished into two types.

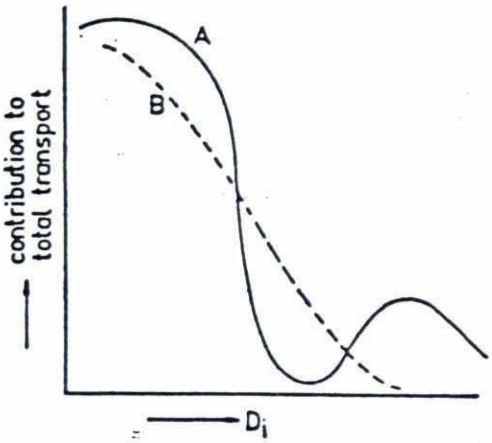

Fig. 1.8. Definition of washload.

In Fig. 1.8 a qualitative plot is given of the contribution of the various fractions (D_i) to the total sediment transport in a river.

River A indicates rivers like the River Rhine, the River Niger and the River Magdalena. A certain range of grain-sizes is hardly present.

On the other hand, however, other rivers like the Serang River on Java do not have such a clear distinction. For type A washload can be characterised by a single grain diameter (50-60 μm). For rivers of type B the grain-size alone cannot be a criterion for the distinction between the two types of transport.

Sieve opening D_i (μm)	150	105	75	62	50	42	35	25	0
$P(D_i)$ (%)	0.9	2.4	4.4	6.9	9.1	11.5	14.4	21.9	100

Table 1.2. Grain-sizes of suspended sedidment (Serang River)

As an example Table 1.2 gives a grain-size analysis of a sediment-water samples taken from the Serang River.

A possible way of distinction seems to be the one indicated independently by Vlugter (1941, 1962) and Bagnold (1962). The energy balance for particles in the stream is considered. Particles require energy to remain in suspension. On the other hand while floating downstream particles deliver potential energy to the stream.

According to this hypothesis the transport of particles with fall velocity W_c becomes unrestricted if

$$\frac{\rho_s - \rho}{\rho_s} W_c \leq u \cdot i \tag{1-1}$$

For quartz ($\rho_s = 2650\ kg/m^3$) this criterion becomes

$$W_c \leq 1.6\ ui \tag{1-2}$$

This *Vlugter-Bagnold criterion* does not only contain the characteristics of the sediment (W_c) but also of the flow (ui). This seems logical: washload is by definition not taking part in morphological processes. If in a river a dam is built with a reservoir then the value of W_c is decreasing in the direction of the dam, according to Eq. (1-2). If the reservoir is large then eventually almost all sediment is trapped, even what was washload in the undisturbed river.

Remark: The data of the Serang River is Table 1.2 show that all (fine) grain-sizes are present. It is a typical example of river type B in Fig. 1.8. The Serang River gets its sediment from the erosion of limestone.

The geological features of the river basin influence the character of a river. The following examples can be given:

- Some rivers have their origin in a lake. For the Nile River the origin of the White Nile is Lake Victoria, whereas the Blue Nile comes from Lake Tana (Ethiopia). Moreover, the discharge of the White Nile is influenced by the swampy area (the Sudds) where much water is lost due to evaporation. The Shire River (Malawi), a tributary of the Zambezi River originates from Lake Malawi.

- Some rivers have a 'rocky section' in their alluvial course. This is for instance the case with the Orinoco River. The Rufiji River in Tanzania has a rocky section at Stiegler's Gorge. At those reaches (nearly) all sediment is transported as washload.

<u>Example</u>: knowledge of the geology of a river is essential for the understanding of the character of the river and the use of a river. A typical example is reported by Neill (1973) as given in Fig. 1.9.

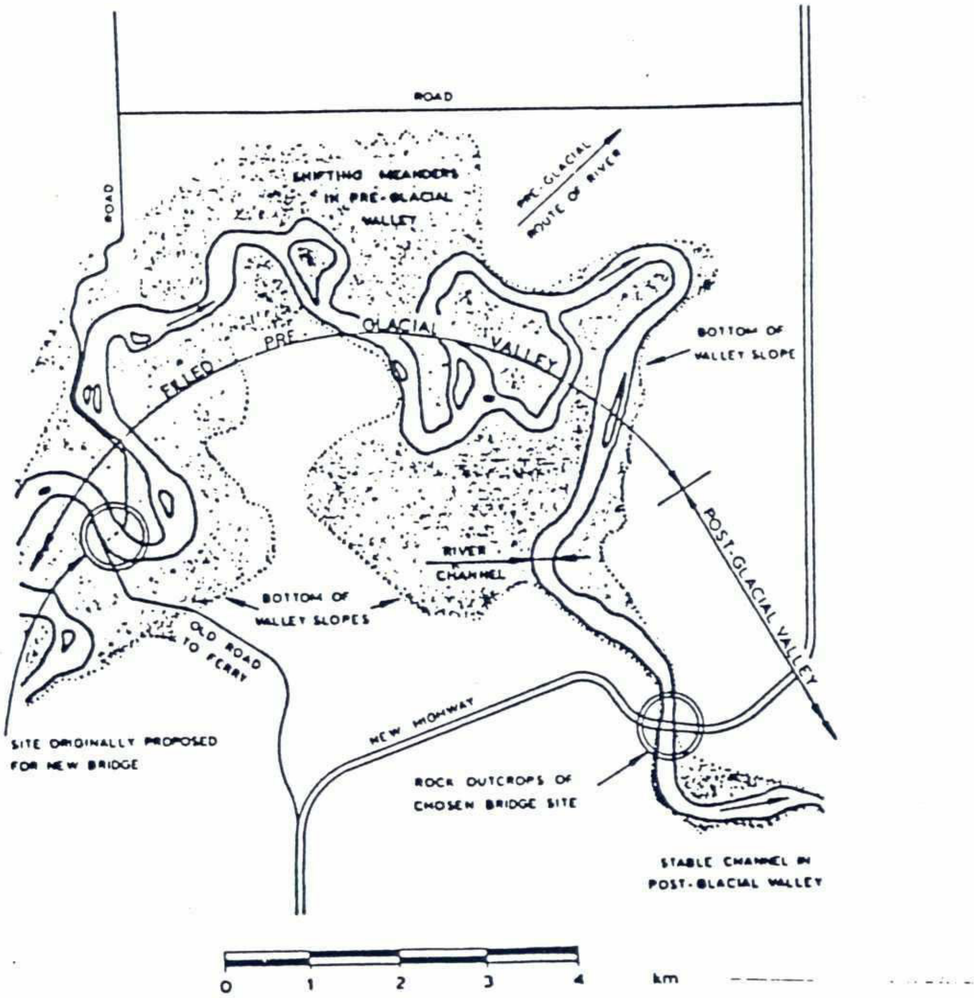

Fig. 1.9. Geology influencing the selection of bridge site (after Neill, 1973).

By studying the geological characteristics of the river valley a bridge site could be selected where it is unlikely that in future the river bed will shift.

1.4. Literature

There exists an abundant amount of literature on potamology and river engineering. Most of it is scattered in articles. Part of it is from a geological (sedimentological) nature, others are directed to river engineering. A few handbooks exist. Scheidegger (1970) has tried from a geomorphological point of view to describe some river processes in a quantitative sense. A mathematical approach to various hydrological aspects of rivers is given by Eagleson (1970). Schumm (1972) and Leopold *et al* (1964) describe some morphological problems.

For the direction of river engineering Shen (1971) and Jansen (1979) can be mentioned. The first book contains a number of separate contributions whereas in the second book an integrated approach is offered. Much information on sedimentation engineering is offered in Vanoni (1975).

Obviously books do not give during a long time the state-of-the-art. For recent finding articles are the appropriate source of information. For instance Jansen (1979) contains material that has mostly been compiled a decade ago from now (1985). The effort spent in the Netherlands on the *research project on rivers* in a close co-operation between Rijkswaterstaat, the Delft Hydraulics Laboratory and the Delft University of Technology during more than one decade has brought forward results that have not yet been incorporated in handbooks. In these lecture notes part of the results are treated.

2. River characteristics

2.1. General

Given from upstream a discharge Q(t) and attached sediment transport S(t) of a grain-size D through a valley with a slope i, a river can have many shapes. Human interference can have altered the shape by major river training (*normalization or canalization*) or by smaller works like local bank protection.This all influences the appearance of a river. Moreover, the river may change its shape as a function of time.
Some general characteristics are treated in this Chapter.

2.2. Planform

In Fig. 2.1 the idealized course of a river is demonstrated. From the *head waters* the river reaches the middle course as a *braided river* gradualy becoming a *meandering river* until in the lower course a delta formation may take place. In the case of a sea (or ocean) the influence of the tides is present in the delta.

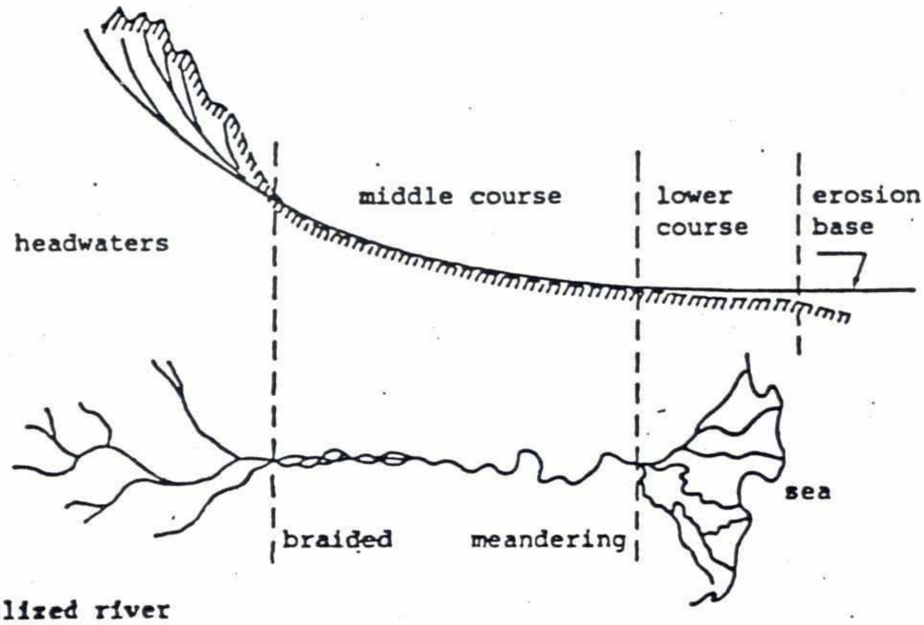

Fig. 2.1. Idealized river

A meandering river is characterized by a single channel whereas a braided river has a number of channels. Leopold and Wolman (1957) have made clear that slope and discharge characterize the planform.

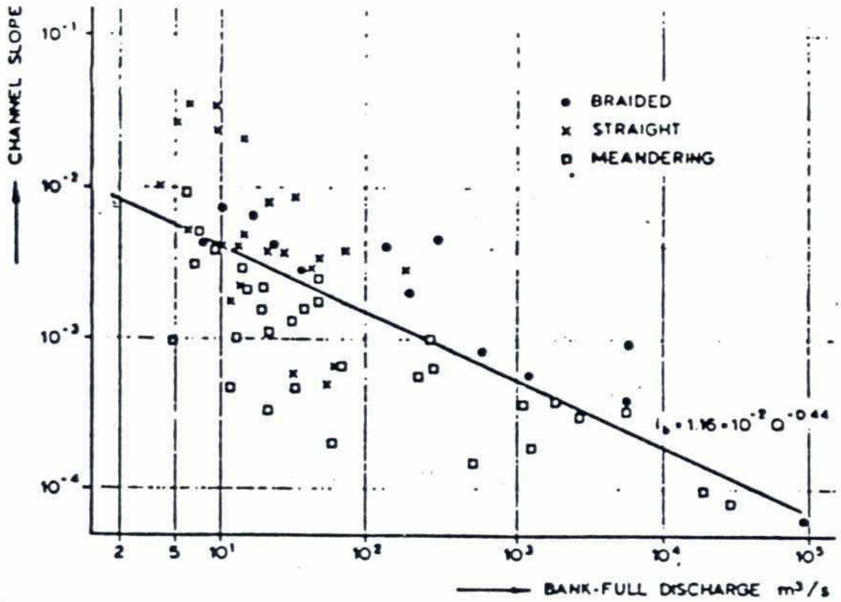

Fig. 2.2. Planform types (after Leopold and Wolman, 1957)

They also mention *straight rivers* as a type of planform. However, this form seems to be unstable. In a straight river there is a tendency to meander in the river bed. There appear *alternate bars* propagating downstream slower than the normal bedforms. These alternate bars have also been noticed in straight laboratory flumes with a mobile bed (Wang and Klaassen, 1981).

The composition of Fig. 2.2 brings forward the problem of schematization of the discharge Q(t) into a single discharge. In Fig. 2.2 the *bankfull discharge* has been taken. It is the discharge just large enough to fill the *low water bed*. Roughly speaking it is the discharge that occurs once or twice in an average year.

Figure 2.3 gives an example of a river in which part is braided and part is meandering. There are indications that the braided part of the Tigris River (the reach upstream of Balad) has an *armoured* bed. Armouring is a result of a degradated bed composed of different grain sizes. Sorting processes are responsible for the fact that finally the toplayer of the bed consists of coarse grains (thickness 1 to 2 D) above the original sediment mixture.

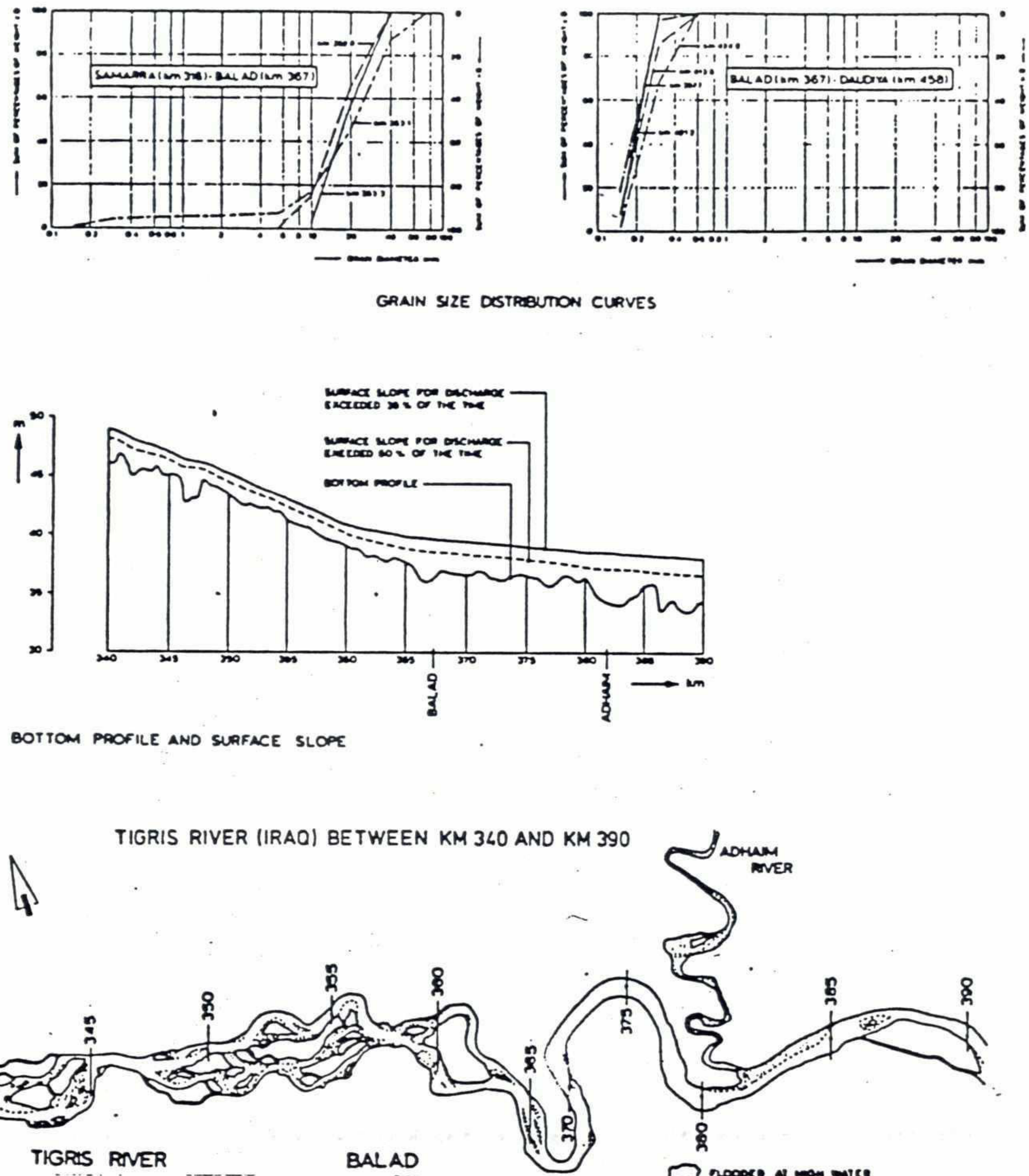

Fig. 2.3. Planform River Tigris (Irak)

Human interference can transform a braided river into a meandering one. An example is given in Fig. 2.4. Normalization works in the 19th century in the River Rhine downstream of Basle (Switzerland) have changed the planform.

The artificial new meanders have fixed banks. If the new course is made straight, alternate bars are likely to occur. This a nuisance for navigation.

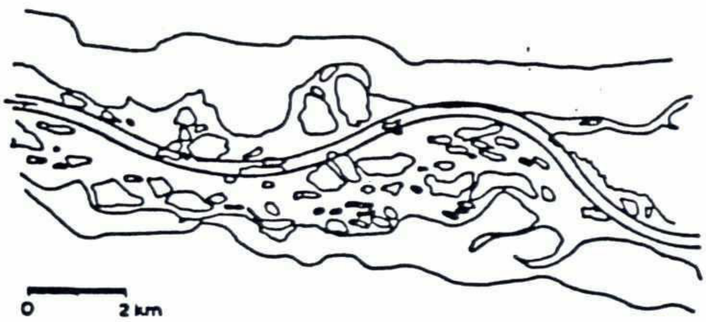

Fig. 2.4. Normalization of the River Rhine downstream of Basle (19th century).

More downstream of Basle the original meandering River Rhine has also been normalized. Figure 2.5 gives an example.

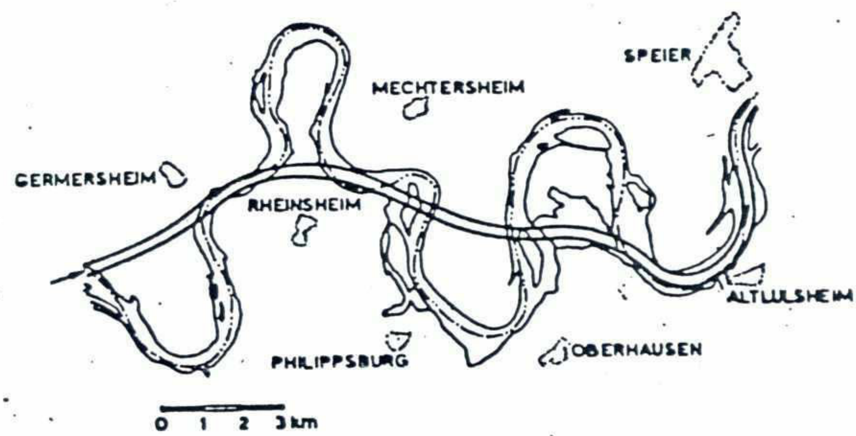

Fig. 2.5. Normalization of the River Rhine upstream of Mannheim (19th century)

In normalized rivers the natural appearance can hardly be recognized. In Fig. 2.6 the change of the River Waal (the main branch of the River Rhine in the Netherlands) in the course of time is represented.

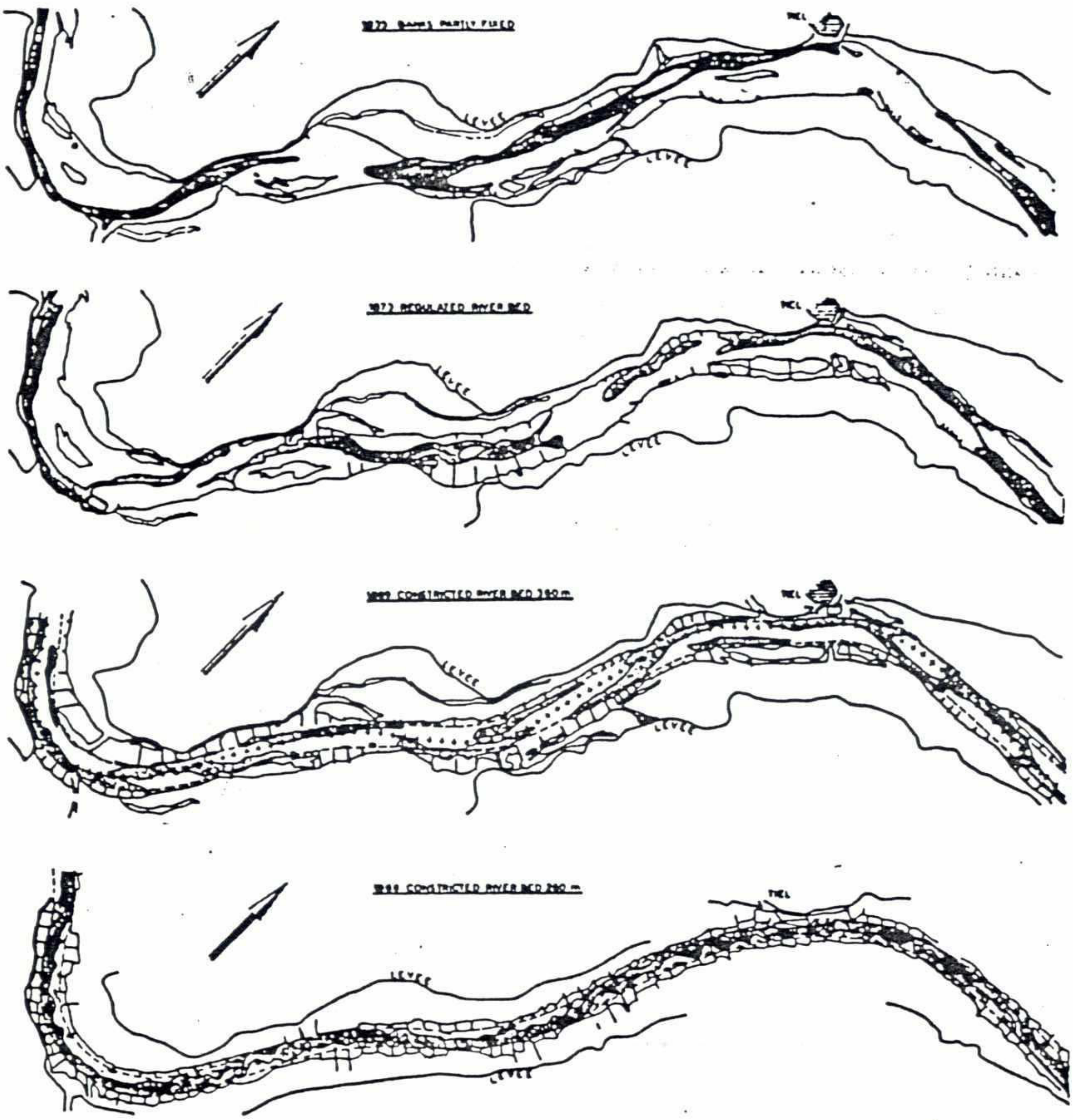

Fig. 2.6. Development of the River Waal near Tiel (km 910 - 930)

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In this case the original reason for river training was the prevention of jamming of flooting ice. Icejams caused flood problems. The later works were carried out to reduce the width to obtain more depth for navigation.

The presence of a meandering or braided rivers has been examined mathematically by means of a linear stability analysis. Some references are given in Janssen (1979, p. 133). The study of Olesen (1983) can be quoted in addition.

The characteristics of meandering rivers have also been studied by many investigators. According to Leopold *et al* (1964) the meander lenght (λ) is roughly proportional to the width (B_s) of the river. The same holds for the relation between λ and the radius of curvature (R_m).
The definitions of the meander characteristics are shown in Fig. 2.7.

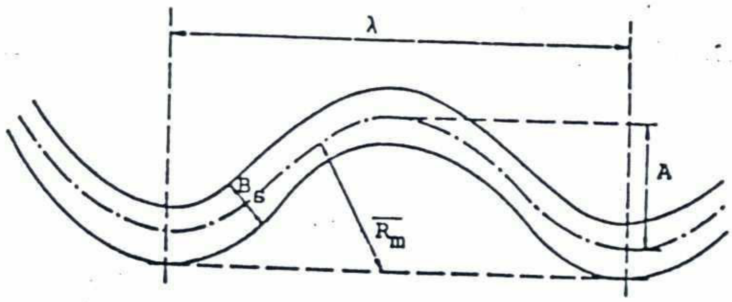


Fig. 2.7. Meander characteristics.

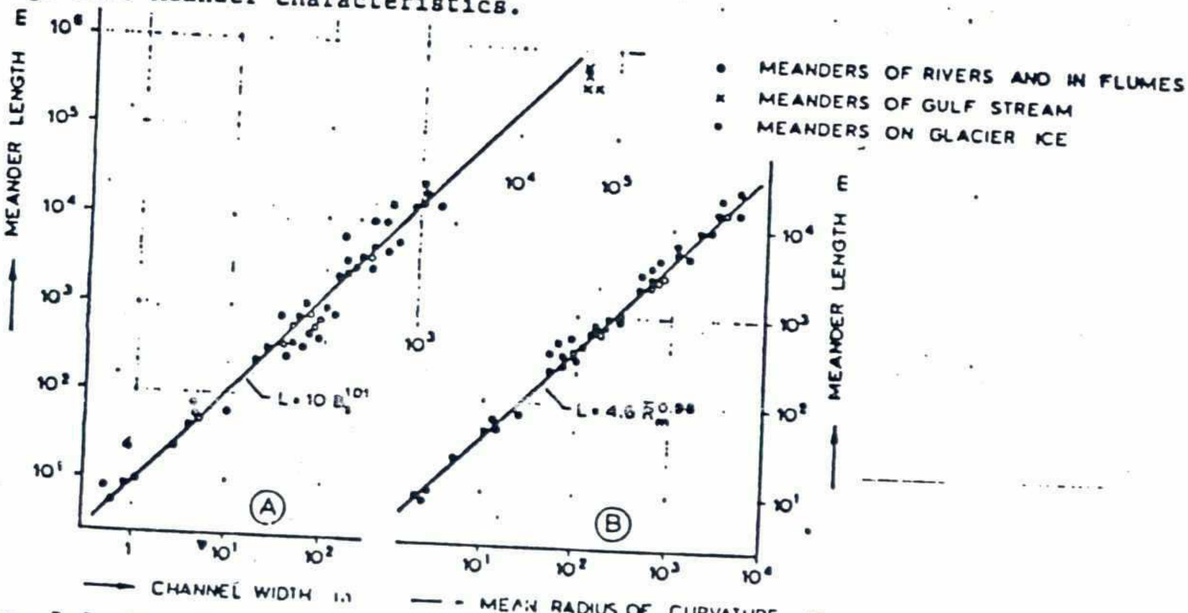


Fig. 2.8. Meander characteristics (after Leopold *et al*, 1964).

The findings of Leopold *et al* (1964) are represented in Fig. 2.8.

The study of meander characteristics is hampered by the fact that not all meanders of the same river are equal. Spectral analysis has been applied by Speight (1965) on the meanders of the Angabunga River (Papua-New Guinea). Fig. 2.9 shows that there are two peaks in the spectra.

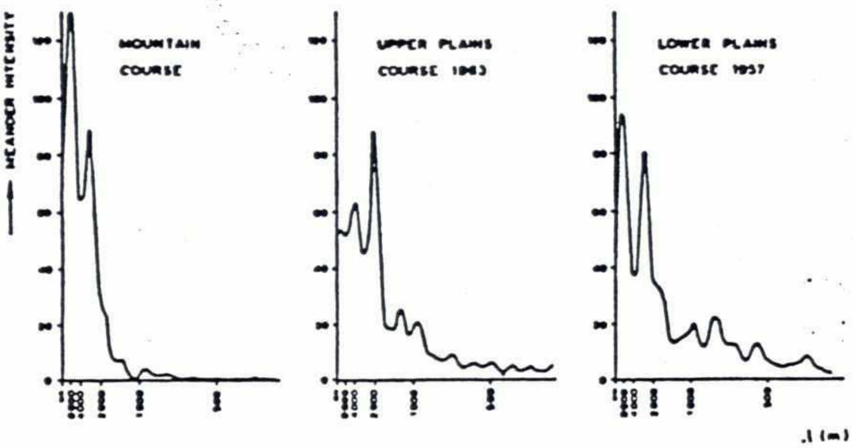


Fig. 2.9. Meander spectra for Angabunga River (after Speight, 1965)

This in accordance with Schumm (1963) who suggests that two characteristic meander lengths may be present for the same stream at the same time.

Another problem is the (varying) discharge. In most cases bankfull discharge has been used to find relation with the mean meander length. A summary given in Jansen (1979, p. 137) suggest $\lambda \propto Q^{\alpha}$ with $\alpha = 0.4$ to 0.5 if the bankfull discharge is taken. Ackers and Charlton (1970) have studied the influence of the hydrograph on the meander length. They studied the River Kaduna (tributary of the Niger River). They tried to reproduce the meanders by means of a scale model and found that reproduction was possible with a constant discharge 13% higher than bankfull discharge.

In freely meandering rivers in time meanders propagate downstream and/or increase their amplitude. If the amplitude becomes very large, the river may during flood cut-off the bend, leaving the original meander loops as *oxbow lakes* in the river valley. Gradually these oxbow lakes get filled up with fine material. This causes inhomogeneities of the sediment composition of the high water bed. Therefore for the Mississippi River the local alignment of the channel depends largely on the local variation of the composition of the bank material (Leopold *et al*, 1964, p. 298).

Hence it is not easy to predict the time depending behaviour of the planform of freely meandering rivers. However, attempts are being made (Ikeda *et al*, 1981; Parker *et al*, 1982 and Chang, 1984).

2.3. Longitudinal profile

The idealized river presented in Fig. 2.1 shows that the bedslope becomes slower in the downstream direction. This is the general tendency found. Moreover, the mean grain-size decreases in the downstream direction. As early as 1875 Sternberg describes this phenomenon mathematically (*see* Leliavski, 1955).

The mass reduction (dM) of the grain during the transport process is supposed to be proportional with the mass (M) of the grains and the distance (dx) over which the grains are transported.
Hence,

$$dM = -\alpha M dx \tag{2-1}$$

in which α is a coefficient describing the properties of the grains and the river.
Integration gives

$$M = M_o \exp\{-\alpha x\} \tag{2-2}$$

in which the integration constant represents the mass at $x = o$

For the grain-size D this can be transformed into

$$D = D_o \exp\{-\alpha' x\} \tag{2-3}$$

The variation of D(x) seems to be due to wearing and sorting. The process has not yet been analysed quantitatively. Leliavski (1955) reports on some data of M{x} for European rivers. Note that in principle the α-value of Eq. (2-2) can have quite different values for rivers. Some times the grain-size can decrease over small distances. This is for instance the case for the Choshui River (Taiwan) as can be noticed from Fig. 1.3.

Also the longitudinal profile can be approached by an experimental function. For the Rio Grande (USA) the relation for the bed slope $i_b = 0.0022 \exp\{-5.8\ 10^{-3}\ x\}$ has been reported. As $i_b = -\partial z_b/\partial x$ also z_b will be changing exponentially with the distance (*see* Jansen, 1979, p. 141).

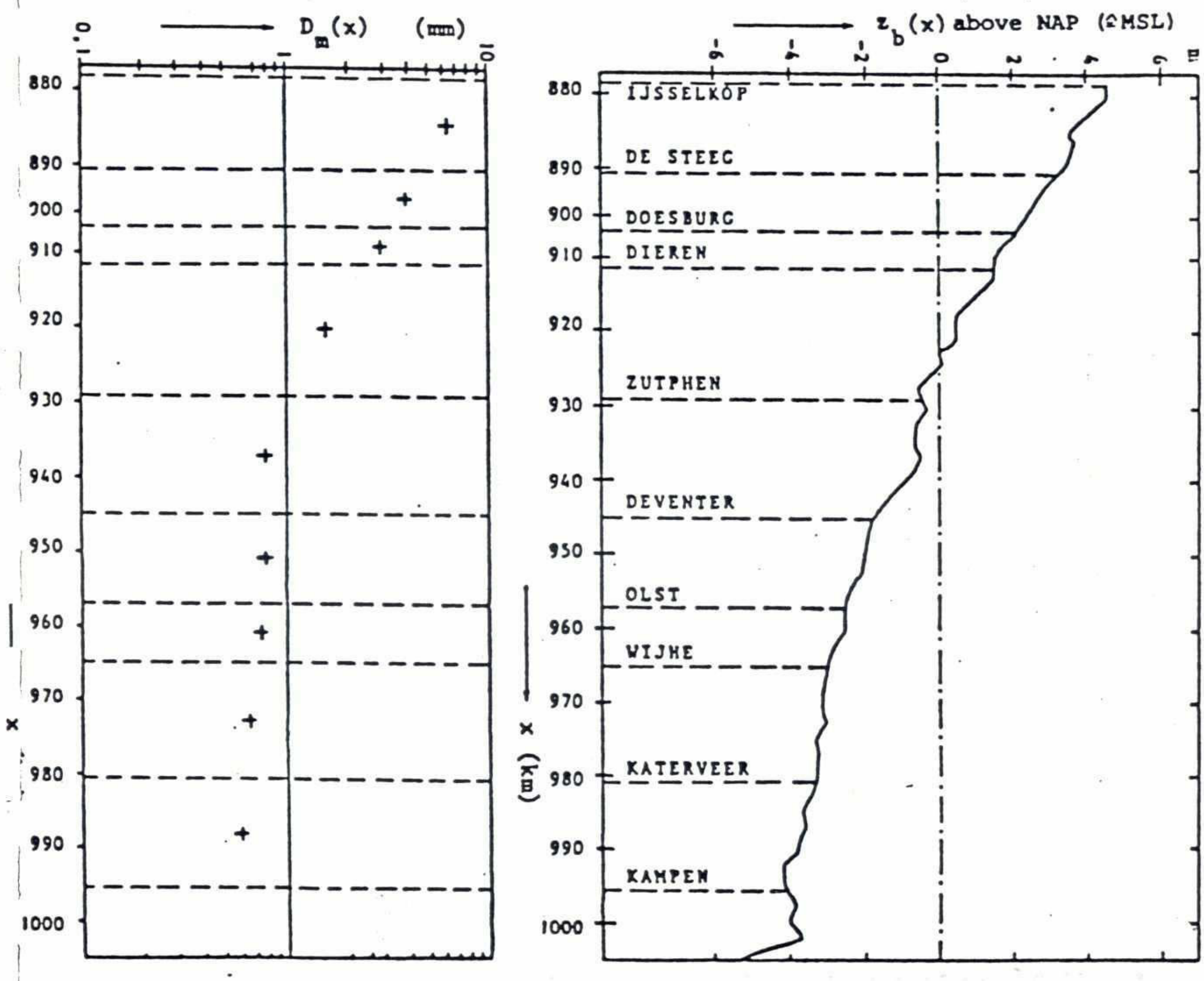


Fig. 2.10. Longitudinal profile IJssel River (after Zeekant, 1983)

2a

As an example of $z_b(x)$ and $\bar{D}(x)$, Fig. 2.10 shows the variation along the axis of the River IJssel, the minor branch of the River Rhine in the Netherlands. Downstream of Kampen the River IJssel is discharging into the IJssel Lake. Hence the downstream part of this river branch is not influenced by tides.

2.4. Confluences and bifurcations

Confluences are mainly present in the upper reach of a river whereas bifurcations are usually present in the lower reach (Fig. 2.1)

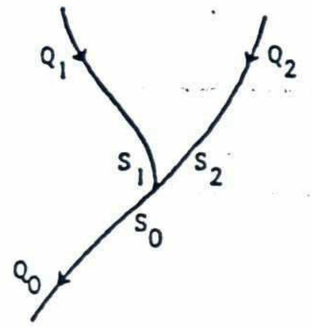


Fig. 2.11. Confluence

For a *confluence* (Fig. 2.11) the equations of continuity for water (Q) and sediment (S) hold.

$$Q_o = Q_1 + Q_2 \quad \text{and} \quad S_o = S_1 + S_2 \qquad (2\text{-}4)$$

The discharges $Q_1(t)$ and $Q_2(t)$ may have a similar shape. This, however, is not always the case. The confluence of the Niger River and the Benue River near Lokoja give an example. Both rivers have a large discharge in September-October ('white flood'). In addition the Niger River has a large discharge in April ('black flood') (*see* NEDECO, 1959). The Niger River mainly governs the water level at the bifurcation. Therefore during the 'black flood' the lower reach of the River Benue contains a backwater curve (M1 - curve). This causes temporarily sedimentation.

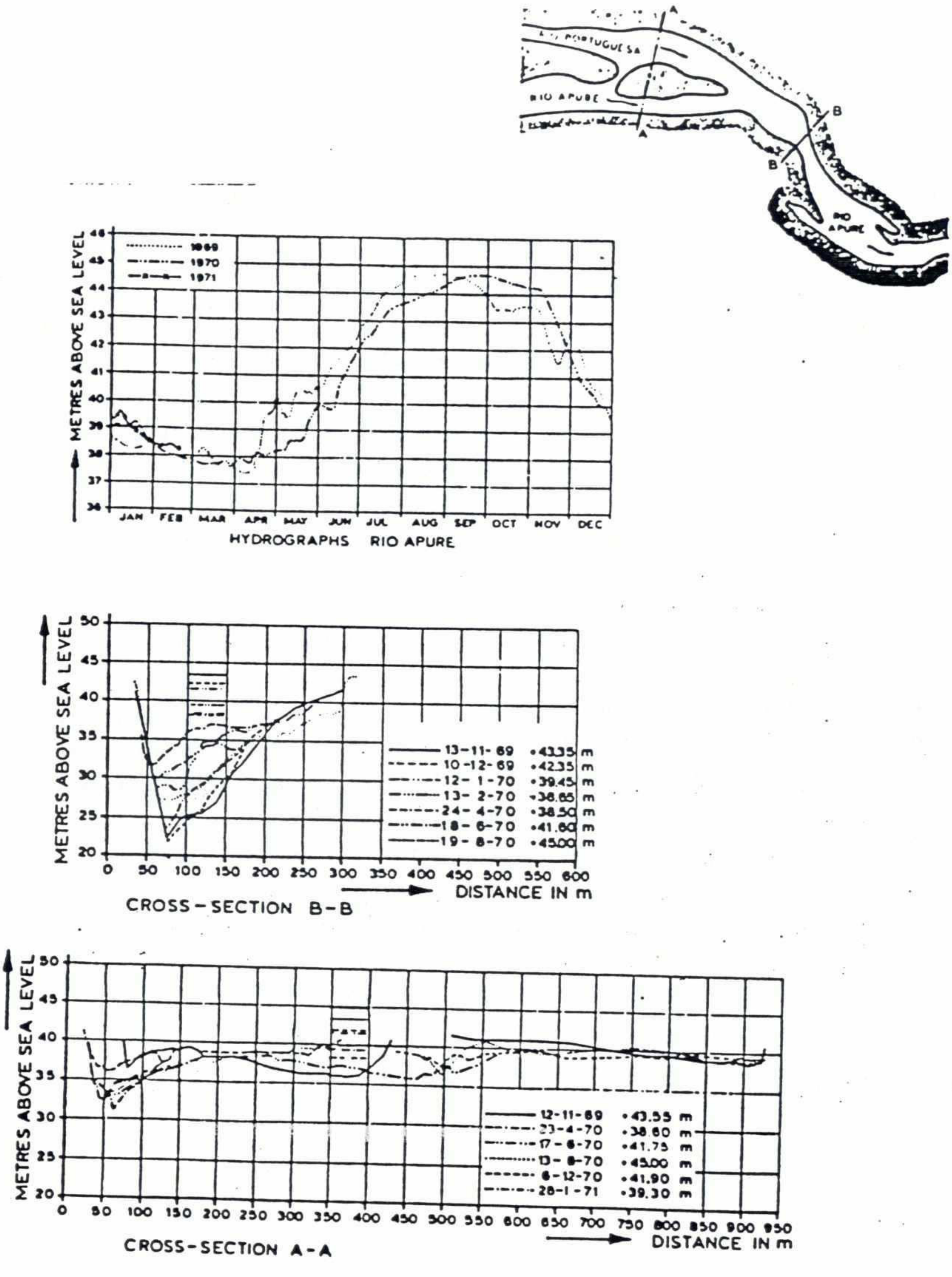


Fig. 2.12. Confluence Apure River and Portugesa River (after DHL, 1971)

Figure 2.12 gives an example of a confluence in Venezuela. The Apure River, near San Fernando de Apure, has a rather regular hydrograph. It regards a tributary of the Orinoco River . Note the large variation of the bed level downstream of the confluence with the Portugesa River, especially in the narrow section B-B. It regards here natural i.e. non-trained rivers.

In the case of a *bifurcation* the discharge (Q_o) and sediment transport (S_o) coming from upstream are devided. (Fig. 2.13).

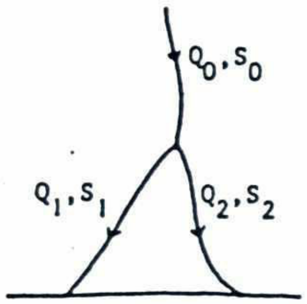


Fig. 2.13. Bifurcation

Of course here also the continuity equations of Eq. (2-4) hold. However, now each equation has two unknowns. Thus additional information is necessary.
This distribution of the discharge Q_o over two branches is governed by the fact that *at* the bifurcation only one water level can exist. Hence the *conveyances* of the two downstream rivers determine the distribution of Q_o.

The distribution of the transport S_o at the bifurcation is more complicated. For some sediment ('washload') the distribution of S_o is proportional to the distribution of Q_o. For the coarse material, transported as bedload, this is not the case. The *local geometry* of the bifurcation determines the local flow pattern and this determines the movement of the sediment transported along the bed. In general a river branching off in an outer bend of another river receives relatively more sediment (Bulle, 1926).

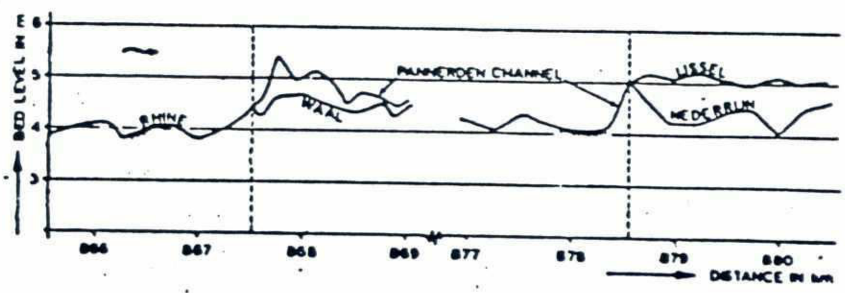


Fig. 2.14. Bed profile near the bifurcations of the River Rhine in the Netherlands

Given the distribution of Q_o and S_o as well as the continuity of the water level at the bifurcation, it is not surprising that the bedlevel can show *discontinuities* (see also Section 4.5). Figure 2.14 shows these discontinuities at the bifurcations of the River Rhine in the Netherlands. The situation of the bifurcations is given in Fig. 2.15.

MEAN VALUES	Q m³/s	S m³/year	D_m mm
RHINE	2140	550	6
WAAL	1490	700	4
PANN. CHANNEL	650	300	4
NEDERRIJN	390	270	2
IJSSEL	360	130	3

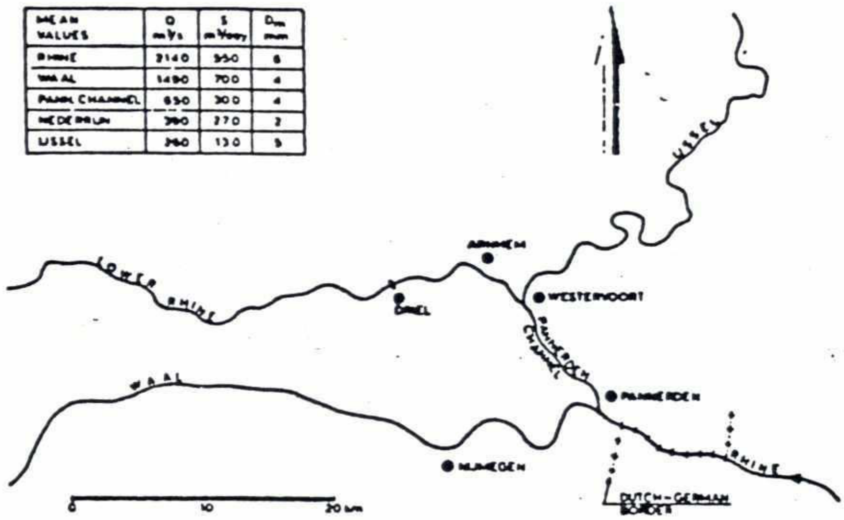


Fig. 2.15. Bifurcations of the River Rhine in the Netherlands

As the sediment coming from upstream is usually non-uniform, grain-sorting is likely to occur at a bifurcation. Figure 2.6 gives an example. It requires some care in sampling to show this phenomenon as other causes of grainsorting (river bends, bedforms) are present at the same time.

To obtain Fig. 2.16 over some kilometers in each branch samples have been taken along the river axis and at distances ± ¼ B from the axis. Each sample had a sufficiently large size to get a good estimation of the mean grain size (D_m) on the particular location (de Vries, 1970).

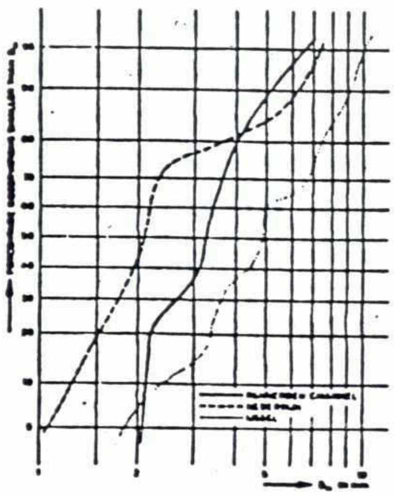


Fig. 2.16. Grain sorting at the bifurcation Westervoort.

2.5. River mouths

A river discharges into another river (like the River Benue into the River Niger), into a lake (like the River IJssel into the IJssel Lake) or in a sea. To a large extent the water level at the mouth is not governed by the river, it is therefore an independent boundary condition. At a far distance upstream of the mouth the water movement and sediment movement are independent of the boundary condition. Naturally the bed level there is influenced by the presence of the mouth.

An elementary analysis of some schematic cases is given below:

- River with constant discharge entering a deep lake

  The most simple case regards a river with constant width and discharge. Upstream of the mouth ($x > 0$) the same S and Q are transported. For uniform bed material the bottom slope (i_b) and the waterdepth (a) will be constant. Waterlevel and bedlevel are then parallel straight lines.

  Due to sedimentation in the lake the mouth will gradually move downstream. The process is governed by the yearly sediment transport and the depth of the lake.

- River with varying discharge entering a deep lake

  At the mouth ($x = 0$) the water level $h(0,t)$ = constant. If again the width (B) and the grain size are supposed to be constant only the variation of Q has to be considered in addition. It can be stated in general that it takes much time to change the slope of the longitudinal profile. Hence the water level upstream of the mouth may vary in time but the bed level hardly does.

  The bed slope (i_b) can now be found from the reasoning that the yearly sediment transport through each cross-section has to be the same. As an approximation the transport formulae for this case are represented by $s = m\,u^n$ with m and n being constant.

The transport can now be expressed with Q and i_b as parameters.

$$S = B\,m\,u^n = B\,m\left(\frac{Q}{Ba}\right)^n = B\,m\left\{\frac{Q}{B}\cdot\left[\frac{Q}{BCi_b^{1/2}}\right]^{-2/3}\right\}^n \qquad (2\text{-}5)$$

in which Chezy equation $Q = BC\sqrt{ai_b}$ is used

Hence

$$S \sim B^{1-n/3} . Q^{n/3} . i_b^{n/3} \qquad (2\text{-}6)$$

If f(Q) is the probability density of the discharge then the yearly sediment transport for each section amounts to

$$\int_0^{\infty} S(Q).f(Q)dQ = \text{constant} \qquad (2\text{-}7)$$

As i_b does not change with the discharge Eqs. (2-6) and (2-7) can be combined.

$$B^{1-n/3}.i_b^{n/3}.\int_0^{\infty} Q^{n/3}. f(Q) . dQ = \text{constant} \qquad (2\text{-}8)$$

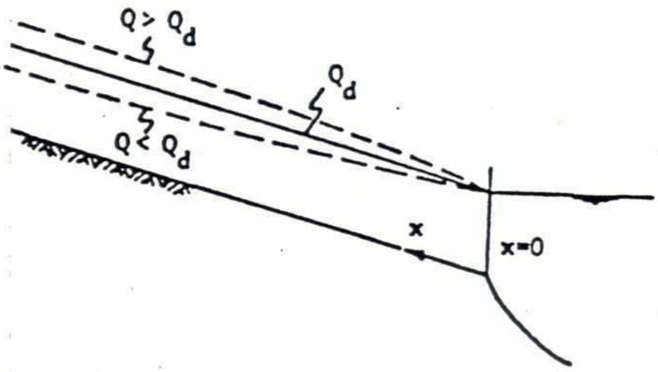


Fig. 2.17. River discharging into a lake

Also in this case i_b is constant (if B is). There is one discharge (Q_d) for which the flow is uniform. For $Q \neq Q_d$ backwater curves are present (Fig. 2.17).

For mild (positive) slopes the backwater curve will be of the M_1-type if $Q<Q_d$ and of the M_2-type for $Q>Q_d$.

It is interesting to look in this case at the depth (a_o) in the mouth. Therefore transport formulae must now be expressed with Q and a as parameters. Combination of Eqs. (2-5) and (2-7) gives for the yearly transport

$$\int_0^{\infty} Bm\left(\frac{Q}{Ba_0}\right)^n.f(Q)\ .dQ = \text{constant} \qquad (2\text{-}9)$$

If now a_o is supposed not to vary with Q (which is less likely than for i_b) then Eq. (2-9) can be written as

$$B^{-n+1} a_o^{-n}\int_0^{\infty} Q^n.f(Q)\ .dQ = \text{constant} \qquad (2\text{-}10)$$

The comparison of Eqs. (2-8) and (2-10) will be given further attention in Section 2.6.

• River discharging into a sea

A river discharging into a sea is near the mouth under the influence of the tidal movement. The tidal movement enlarges the sediment transporting capacity. Therefore the cross-section will generally increase in the direction of the mouth. The principle can be explained from the non-linear relationship between flow velocity and sediment transport.

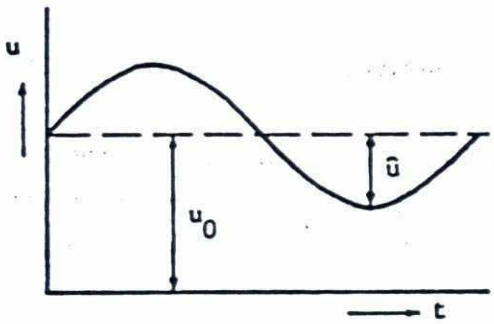


Fig. 2.18. Tidal influence

In Fig. 2.18 the variation of the flow velocity is given. Due to upstream discharge (Q_o) there is the flow velocity u_o.

The flow velocity due to the tide is supposed to vary as a sine-function with an amplitude û. Therefore the flow velocity in the cross-section considered reads

$$u = u_o + \hat{u} \sin \omega t \qquad (2\text{-}11)$$

The transport per unit of width is s and using $s = m\,u^n$ gives for the average transport $\bar{s}$ during the tidal period (T):

$$\bar{s} = T^{-1} \int_o^T m\,\{u_o + \hat{u} \sin \omega t\}^n\, dt \qquad (2\text{-}12)$$

If the parameters m and n do not change too much this gives with $\phi = \hat{u}/u_o$

$$\bar{s} = m u_o^{\,n} \cdot T^{-1} \int_o^T \{1 - \phi \sin\omega t\}^n\, dt \qquad (2\text{-}13)$$

Due to the upper discharge Q_o the transport would be $s_o = m\,u_o^{\,n}$.

From Eq. (2-13) follows with $\omega t = 2\pi$ and $\omega t = y$ or $dy = \omega dt$:

$$\bar{s} = \bar{s}_o \cdot \frac{1}{2\pi} \int_0^{2\pi} \{\phi \sin y + 1\}^n\, dy = \beta s_o \qquad (2\text{-}14)$$

with

$$\beta = \frac{1}{2\pi} \int_0^{2\pi} (\phi \sin y + 1)^n \, dy = \beta(\phi, n) \qquad (2\text{-}15)$$

For instance the following functions for $\beta(\phi)$ can be found analytically:

$n = 1 \quad \beta = 1$

$n = 3 \quad \beta = 3/2\,\phi^2 + 1$

$n = 5 \quad \beta = 15/8\,\phi^4 + 5\phi^2 + 1$

These relations are given in Fig. 2.19 for $0 \leq \phi \leq 1$.

Due to above mentioned non-linear relationship $n > 1$. Thus $\beta > 1$ or $\bar{s} > s_o$.

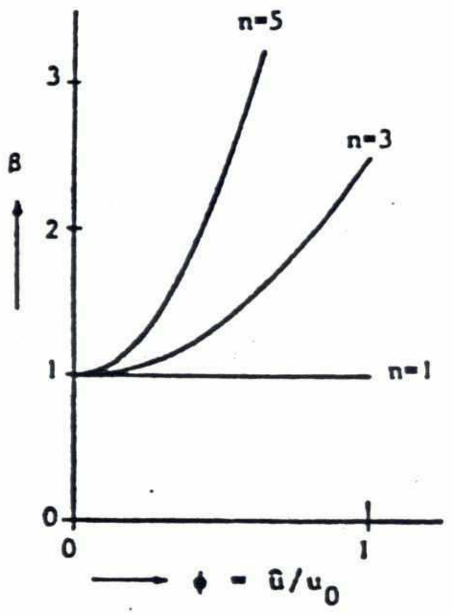


Fig. 2.19. $\beta = f(\phi, n)$

Consider now two cross-sections. The one upstream of the tidal influence (subscript o) has the characteristics $u = u_o$; $\phi_o = 0$ and $B = B_o$. The cross-section under the tidal influence has the subscript 1. So here the characteristics are $\phi = \phi_1$ and $B = B_1$.

For a constant upper discharge Q_o the mass balance has to express that both cross-sections have to have the same total transport as $S_o = S_1$ or

$$B_1 \, \beta_1 \, . s_{o1} = B_o \, . \, s_{oo} \qquad (2\text{-}16)$$

in which s_{o1} is the transport unit width in the cross-sectional area A_1 with $Q_o = u_{o1} . A_1$

From Eq. (2-16) follows:

$$B_1 \cdot \beta_1 \cdot m \left[\frac{Q_o}{a_1 B_1}\right]^n = B_o \cdot m \left[\frac{Q_o}{a_o B_o}\right]^n \qquad (2\text{-}17)$$

Or

$$\beta_1 = \frac{B_1^{\,n-1} \cdot a_1^{\,n}}{B_o^{\,n-1} \cdot a_o^{\,n}} = \frac{B_o}{B_1}\left[\frac{A_1}{A_o}\right]^n \qquad (2\text{-}18)$$

As $\beta_1 > 1$ for a constant width ($B_1 = B_o$) it follows from Eq. (2-18) that $a_1 > a_o$. In general it will follow from Eq. (2-18) that $A_1 > A_o$.

The above given analysis is only of a qualitative nature. Near the mouth the analysis will not hold due to the fact that density currents will be present and the flow direction will reverse.

2.6. Schematization of the regime

The main characteristic of a river discharge is that this varies in time. As a consequence the morphological parameters of a river will also be time-depended. Therefore if morphological forecasts have to be made, this variation in time has to be taken into consideration.

At present (1985) it has become possible to carry out these morphological computations with a varying discharge $Q(t)$. However, it is then still questionable which (recorded) $Q(t)$ has to be taken. There will be a tendency to use an average year and if possible also wet years will be used. No systematic research as yet seems to have been carried out.

Instead of a time depending prediction it is possible to study the change of an equilibrium situation into a new one, leaving the time depending predictions of the transient from one equilibrium into another for a second approximation. In this steady approach the probability distribution $f(Q)$ is used. This method has been used in Section 2.5 to find the equilibrium bed slope of a river discharging into a lake. In Eq. (2-8) the right hand side represents the yearly sediment transport. Hence this equation can be used to study the change of the slope if the width of the river is changed (see also Section 4.4).

It has to be remarked that in the above quoted analysis the transport function is approached by a power law $s = m\, u^n$. This has been done to make the analysis sufficiently transparent. For practical problems it is quite possible to use a real transport formula, i.e. one adequate to the river which is studied.

In literature frequently the river regime is drastically schematized into one single discharge (*'dominant discharge'*). The use of bankfull discharge for the study of meander characteristics is an example (Section 2.2).
It can easily be shown that such a dominant discharge does not exists. In other words one single discharge cannot describe more than one morphological parameter of a river.

The proof of this statement can be obtained from the example of a river discharging into a lake (see Fig. 2.17). Two parameters are considered *viz* the bed slope i_b upstream of the mouth and depth a_o at the mouth.
Following the procedure usually applied with the concept of 'dominant discharge' Eq. (2-8) would lead to a discharge Q_{d_i} for the slope i_b according to

$$B^{1-n/3} \cdot i_b^{\,n/3} \int_0^\infty Q^{n/3} \cdot f\{Q\}\, dQ = B^{1-n/3} \cdot i_b^{\,n/3} \cdot Q_{d_i} \qquad (2\text{-}19)$$

or

$$Q_{d_i}^{\,n/3} = \int_0^\infty Q^{n/3} \cdot f\{Q\} \cdot dQ \qquad (2\text{-}20)$$

A similar approach for the depth a_o would lead with Eq. (2-10) to a 'dominant' discharge Q_{d_a} with

$$Q_{d_a}^{\,n} = \int_0^\infty Q^n \cdot f\{Q\} \cdot dQ \qquad (2\text{-}21)$$

Equations (2-20) and (2-21) show that always $Q_{d_i} \neq Q_{d_a}$. In other words one single discharge *cannot* lead to correct answers for both i_b *and* a_o.

Two more remarks can be made in this respect.

(1) The definitions applied to find the 'dominant' discharge use the characteristics of the *existing* river. Obviously a different discharge has to be applied to forecast the response of the river on man-made changes in the river system.

(ii) The above given examples for i_b and a_o show that there is no need to define such a thing as a 'dominant' discharge. In principle the problem of finding i_b and a_o can be solved by means of Eqs. (2-8) and (2-10).

In summary the schematization of the regime of a river can be two-fold.

- For time-depending prediction the 'real' Q(t) has to be used.
- For studying new equilibrium situations it is advised to use the probability density f{Q} of the discharge.

In practice both f{Q} and Q(t) will be approximated. For instance

$$\int_0^{\infty} f\{Q\}\, dQ = \sum_{i=1}^{n} Q_i \qquad (2\text{-}22)$$

As an example it can be tested whether a continuous probability density f{Q} based on daily discharges can be approximated by a histogram based on monthly averaged discharges. As the sediment transport plays a key role in the morphological predictions it is logical to test this approximation via S. This can be done by some test computations of the factor α with n being the number of days in a month and

$$\alpha = \frac{n * S\{\bar{Q}_{month}\}}{\sum_{i=1}^{n} S\{\bar{Q}_{day}\}} \qquad (2\text{-}23)$$

For flushy upper rivers due to the non-linear relationship between Q and S the value will be $\alpha \ll 1$. For lower rivers, however, the discharge usually does not change rapidly. Then $\alpha \simeq 1$, which means that Eq. (2-22) can be used thus the computations can be based on monthly averaged discharges.

A similar approach may be used for time-depending morphological computations with Q(t). As will be shown later (see Chapter 3) in morphological computations often time steps larger than one day can be used. Hence also in that case discretization is adopted, this time of Q(t).

3. Fluvial processes

3.1. General

The combined transport of water and sediment in rivers is a complex process because there is an interaction between the transports of the two phases. The problem is time-depending, dealing with three space dimensions. It requires a great deal of schematization in order to be able to describe the problems in a mathematical sense, leading to mathematical models that can be used for morphological forecasts.

In this chapter the mathematical description is treated. In the first place Section 3.2 deals with the one-dimensional approach. Here the average values of the morphological parameters for each cross-section are considered as function of time and place. In this approach there is only one space dimension left, the coordinate x along the river axis.

In Section 3.3 two-dimensional approaches are treated. The two space dimensions are in the first place the x and y coordinate in the horizontal plane. Also two-dimensional approaches in the x-z plane are considered (Sub-Section 3.3.3). These approaches are necessary when the transport of sediment in suspension varies considerably in the longitudinal direction.

The basic parameters are indicated in the definition sketch of Fig. 3.1.

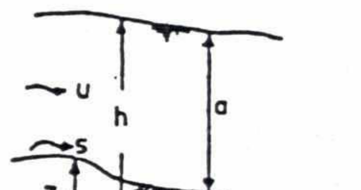


Fig. 3.1. Definition sketch.

- The *water depth* (a) is mainly of importance for navigation. Prediction of a (x,y,t) is anticipated.
- The *water level* (h) is of interest for the possibility to withdraw water for irrigation or with regard to flood problems.
- The *bed level* (z_b) is important to know when bank protection works or bridge piers have to be designed. Obviously $z_b(x,y,t)$ has to be predicted.

3.2. One-dimensional approach

3.2.1. Analysis of basic equations

In the one-dimensional approach the average values of a, h, and z_b are considered for the cross-sections. With $h = a + z_b$ according to the definition (Fig. 3.1) this means that a and z_b can be considered as dependent variables for which relevant basic equations have to be found. Moreover the flow velocity u(x,t) and the transport s(x,t) are dependent variables. This means that *four* basic equations are required.

The equations are:

momentum water $$\frac{\partial u}{\partial t} + u\frac{\partial u}{\partial x} + g\frac{\partial a}{\partial x} + g\frac{\partial z_b}{\partial x} = -g\frac{u|u|}{C^2 a} \qquad (3\text{-}1)$$

continuity water $$\frac{\partial a}{\partial t} + u\frac{\partial a}{\partial x} + a\frac{\partial u}{\partial x} = 0 \qquad (3\text{-}2)$$

transport formula $$s = f\{u, \Delta, D, C \text{ etc}\} \qquad (3\text{-}3)$$

continuity sediment $$\frac{\partial z_b}{\partial t} + \frac{\partial s}{\partial x} = 0 \qquad (3\text{-}4)$$

The following remarks have to be made:

(i) The equations are valid for a wide river with constant width B. The banks are supposed to be fixed or less erodible than the river bed. For erodible banks also B(x,t) would have to be considered as a dependent variable. This would require an additional equation, which is not readily available.

(ii) The equations are valid for $s/q \ll 1$; i.e. small mean sediment concentrations.

(iii) Any suitable transport formula can be used in principle. In this elementary analysis all parameters except u(x,t) are supposed not to vary with x and t.

(iv) Equation (3-3) implies that the sediment transport is a function of the *local* hydraulic parameters. Hence this model is not applicable if there is a change in suspended load over short distances (see Sub-Section 3.3.3).

It has been shown (de Vries, 1959, 1965) that Eqs. (3-1) through (3-4) form a hyperbolic system with the characteristic celerities $dx/dt = c$. The three celerities $c_{1,2,3}$ are the roots of the cubic equation:

$$c^3 - 2uc^2 - (ga - u^2 + gdf/du)c + ugdf/du = 0 \qquad (3\text{-}5)$$

An analysis can for instance be found in Jansen (1979, p. 94).
Equation (3-5) can be modified using the following three dimensionless parameters.

$$\begin{aligned} &\bullet \text{ relative celerity} && \phi = c/u \\ &\bullet \text{ Froude number} && Fr = u/\sqrt{ga} \\ &\bullet \text{ Transport parameters} && \psi = a^{-1}df/du \end{aligned} \qquad (3\text{-}6)$$

The dimensionless form of Eq. (3-5) becomes then:

$$\phi^3 - 2\phi^2 + (1-Fr^{-2} - \psi Fr^{-2}) + \psi Fr^{-2} = 0 \qquad (3\text{-}7)$$

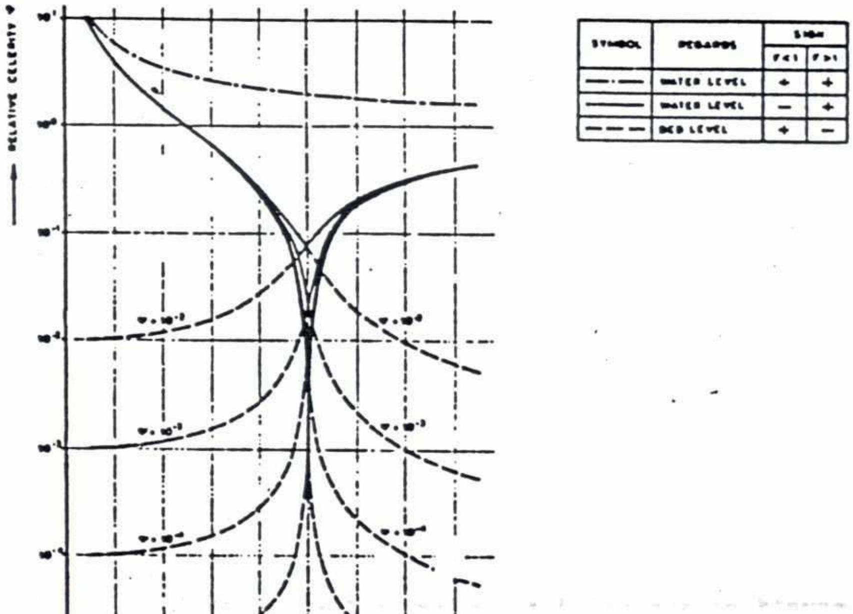


Fig. 3.2. Relative celerity of disturbance (after de Vries, 1969)

In Fig. 3.2 the three roots c_i ($i = 1,2,3$) have been represented graphically as functions of the Froude number and ψ.

Before analysing this figure it is of importance to pay some attention to the parameter ψ.
Using as an approximation $s = m\,u^n$ it follows

$$\psi = \frac{df/du}{a} = \frac{m\,n\,u^{n-1}}{a} = n\,\frac{s}{q} \qquad (3\text{-}8)$$

Hence $\psi \sim s/q$, a value which is usually much smaller than unity. Note that $O(n)=O(5)$. Therefore in Fig. 3.2 only values $\psi \ll 1$ are sketched. The figure shows that two celerities $c_{1,2} = u \pm \sqrt{ga}$ or $\phi_{1,2} = 1 \pm Fr^{-1}$ are apparently for $Fr < 0.6$ not influenced by the mobility of the bed (thus by ψ).

Inserting for this case the known roots $\phi_{1,2}$ in Eq. (3-7) lead to an expression for ϕ_3. This can be done as follows. Equation (3-7) can be written as:

$$(\phi - \phi_1)(\phi - \phi_2)(\phi - \phi_3) = 0 \qquad (3\text{-}9)$$

So comparing Eqs. (3-7) and (3-9) gives

$$\psi\,Fr^{-2} = -\,\phi_1\phi_2\phi_3 \qquad (3\text{-}10)$$

or

$$\psi\,Fr^{-2} = -(1+Fr^{-1})(1-Fr^{-1})\,\phi_3 \qquad (3\text{-}11)$$

thus

$$\phi_3 = \frac{\psi}{1 - Fr^2} \qquad (3\text{-}12)$$

Note that in Fig. 3.2 for $Fr < 0.6$ it holds $|\phi_{1,2}| \gg \phi_3$ Hence if we are interested in changes of the bed the Eq. (3-12) can be used for ϕ_3, moreover $|\phi_{1,2}| \to \infty$ can be concluded. This implies that the flow can be assumed to be quasi steady. Thus for this case Eqs. (3-1) through (3-4) can be simplified to

$$u \frac{\partial u}{\partial x} + g \frac{\partial a}{\partial x} + g \frac{\partial z_b}{\partial x} = -g \frac{u|u|}{C^2 a} \qquad (3\text{-}13)$$

$$a \frac{\partial u}{\partial x} + u \frac{\partial a}{\partial x} = 0 \quad \text{or} \quad q = q(t) \qquad (3\text{-}14)$$

$$s = f(u, \text{parameters}) \qquad (3\text{-}15)$$

$$\frac{\partial z_b}{\partial t} + \frac{\partial s}{\partial x} = 0 \qquad (3\text{-}16)$$

Thus for $Fr < 0.6$ the system of equations can be decoupled. Equations (3-13) and (3-14) can be combined to the equation for the backwater curve

$$\frac{\partial u}{\partial x} \left[u - \frac{gq}{u^2} \right] + g \frac{\partial z_b}{\partial x} = - g \frac{u^3}{C^2 q} \qquad (3\text{-}17)$$

For a given discharge q and known bed level z_b the flow velocity u can be computed for specific boundary conditions.

Moreover Eqs. (3-15) and (3-16) can be combined into:

$$\frac{\partial z_b}{\partial t} + \frac{df(u)}{du} \cdot \frac{\partial u}{\partial x} = 0 \qquad (3\text{-}18)$$

Thus for known velocities u the bed level in future can be computed with Eq. (3-18) if the appropriate boundary conditions are applied.
Hence Eqs. (3-17) and (3-18) in principle can be used for the description of morphological processes in rivers.
Two additional assumptions can lead to a further simplification of Eqs. (3-17) and (3-18). This leads to two mathematical models that can be used for analysing morphological phenomena.

(i) For *small values* of x and t the *friction term* (right hand side) in Eq. (3-17) can be neglected with respect to the other terms. This gives the *simple wave model*.

(ii) For *large values* of x and t the *backwater effects* (first terms in Eq. (3-17)) can be neglected. This leads to the *parabolic model*.

ad (i) The characteristics of the *simple wave model* can be demonstrated easily when in addition the assumption $Fr \ll 1$ is made. The momentum equation (Eq.(3-13)) simplifies then into

$$\frac{\partial a}{\partial x} + \frac{\partial z_b}{\partial x} = \frac{\partial h}{\partial x} = 0 \quad \text{thus } h = \text{const.} \qquad (3\text{-}19)$$

This means that the water level is horizontal. (*'rigid lid approximation'*)

As $q = u.a$ = constant, it can be written $u\, \partial a/\partial x + a\partial u/\partial x = 0$
Combining this with Eq. (3-18) yields

$$\frac{\partial z_b}{\partial t} + \frac{df(u)}{du} \cdot \left[- \frac{u\partial a}{\partial x} \right] = 0 \qquad (3\text{-}20)$$

Considering in addition Eqs. (3-8) and (3-12) for $Fr^2 \ll 1$ gives

$$\frac{\partial z_b}{\partial t} - c\frac{\partial a}{\partial x} = 0 \qquad (3\text{-}21)$$

From Eq. (3-19), however, follows $\partial a/\partial t = -\partial z_b/\partial t$. Hence

$$\frac{\partial a}{\partial t} + c\frac{\partial a}{\partial x} = 0 \qquad (3\text{-}22)$$

An application of this simple-wave equation is given in Sub-Section 3.2.2.

ad (ii) The *parabolic model* is obtained from Eq. (3-17) if the first term (responsible for the backwater effects) is neglected.
Differentiation with respect to x gives

$$\frac{\partial^2 z_b}{\partial x^2} + 3\frac{u^2}{C^2 q}\frac{\partial u}{\partial x} = 0 \qquad (3\text{-}23)$$

Eliminating $\partial u/\partial x$ from Eqs. (3-18) and (3-23) gives

$$\frac{\partial z_b}{\partial t} - \left[\frac{df(u)}{du} \cdot \frac{1}{3} \cdot \frac{Cu^2}{q}\right]\frac{\partial^2 z_b}{\partial x^2} = 0 \qquad (3\text{-}24)$$

Linearization yields

$$\frac{df(u)}{du} \cdot \frac{1}{3} \cdot \frac{Cu^2}{q} = \frac{df(u)/du}{di/du} \approx \frac{ds_0/du}{di_0/du} = K \qquad (3\text{-}25)$$

The parabolic model gives therefore the following morphological equation to describe the changes of the bedlevel

$$\frac{\partial z_b}{\partial t} - K \frac{\partial^2 z_b}{\partial x^2} = 0 \qquad (3\text{-}26)$$

with

$$K = \frac{ds_o/du}{di_o/du} = \frac{mnu^{n-1}}{1/3u^2/C^2q} = \frac{3ns_o}{i_o} \qquad (3\text{-}27)$$

in which the subscript o refers to the orginal uniform situation for which changes are considered.

Remarks:

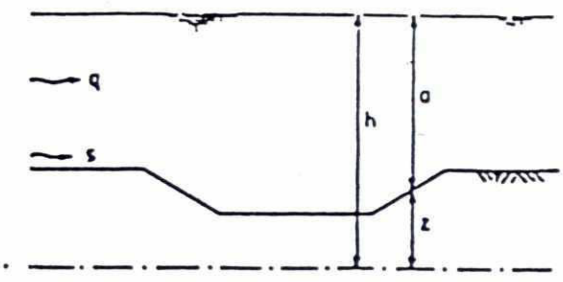
- In the above derivation only differentiation with respect to x has been applied. Therefore the derivation remains valid for $q = q(t)$ and $s = s(t)$. For a river with varying discharge $K = K(t)$. Hence the general equation reads

$$\frac{\partial z_b}{\partial t} - K(t) \frac{\partial^2 z_b}{\partial x^2} = 0 \qquad (3\text{-}28)$$

- In the derivation backwater effects have been neglected. This is only valid if relatively large values of x are considered. Comparison with the complete equation leads to the conclusion that the assumption implies the condition $\Lambda > 2$ to 3 with $\Lambda = xi_o/a_o$. The 'length scale' Λ is a characteristic parameter for the river considered. Note that $\Lambda = 1$ is obtained for a river reach with a length over which the difference in piezometric head is equal to the normal depth.

An application of the use of Eq. (3-18) is given in Sub-Section 3.2.2.

3.2.2. Example: Deformation of a dredged trench



In Fig. 3.3 a trench is represented dredged across a river (t=0).
How will the trench be deformed if only bedload transport is present?
It has to be noted that relatively small values of x are concerned. Hence the *simple wave equation* can be applied.

Fig. 3.3. Dredged trench.

The variation in the depth a is so large that the celerity c cannot be considered constant.
If the variation in depth is considered it follows from Eq. (3-20) with $s = m\,u^n$

$$\frac{\partial z_b}{\partial t} - \left[m\ n\ u^n \right] a^{-1} \frac{\partial a}{\partial x} = 0 \qquad (3\text{-}29)$$

Or, for a constant discharge and a horizontal water level $\partial z_b/\partial t = -\partial a/\partial t$ thus

$$\frac{\partial a}{\partial t} + \left[\frac{m\ n\ q^n}{a^{n+1}} \right] \frac{\partial a}{\partial x} = 0 \qquad (3\text{-}30)$$

Thus

$$\frac{\partial a}{\partial t} + c(a)\ \frac{\partial a}{\partial x} = 0 \qquad (3\text{-}31)$$

Now the deformation of the trench can be estimated qualitatively for $t > 0$.
Three parts can be considered:

- The *horizontal bed* will not deform as $\partial a/\partial x = 0$, hence $\partial u/\partial x = 0$ thus $\partial s/\partial x = 0$ and therefore $\partial z/\partial t = 0$.
- The *downstream slope* will become flatter because $\partial a/\partial x < 0$ thus $\partial u/\partial x > 0$ or $\partial s/\partial x > 0$ and $\partial c/\partial x > 0$. A point of the slope with depth a will move in the time Δt downstream over a distance $\Delta x = c(a)\,\Delta t$.
- The *upstream slope* will for $t > 0$ get steeper. This will continue until the slope will be under the angle of repose.

In Fig. 3.4 the situation is sketched for t = 0 for t = Δ t.

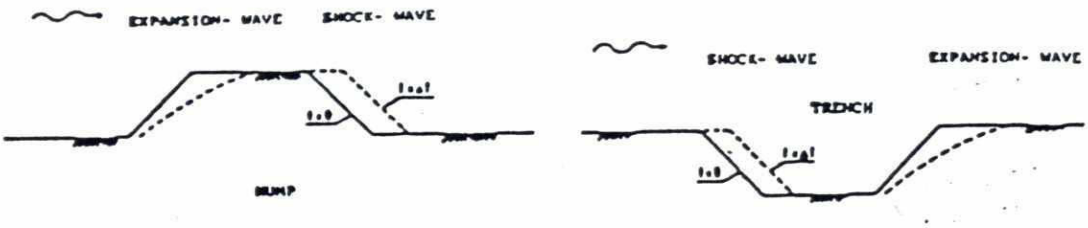


Fig. 3.4. Deformation of a hump and a trench.

In this figure also the deformation of a hump instead of a trench is given. Note the similarity and the difference. A gradual flattening of a slope is similar to what is called in gasdynamics an *expansion wave*. The opposite (the slope becomes steeper) is called a *shock wave*.

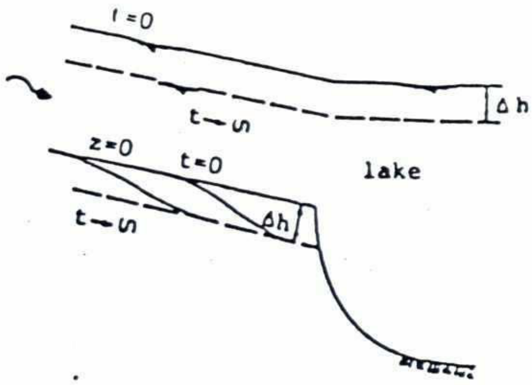
Remarks:

(i) For the above given considerations it is essential that s = f(u) holds. For small distances this implies that bedload transport is postulated. In the case of suspended load the picture can be completely different.

(ii) The information on the deformation of a hump will be used to explain the morphological phenomena occuring due to closure of one branch of a river flowing around an island (Section 4.6).

3.2.3. Example: Morphological time-scale for rivers

The parabolic model derived in Sub-Section 3.2.1 can be used to obtain some information on the speed at which morphological processes in rivers take place. A *morphological time-scale* can be defined (de Vries, 1975).



It is assumed that the river considered is discharging into a lake. At t=0 the water level of this hypothetical lake is lowered over a distance h. This leads to a degradation of the riverbed which ends at t→∞. When also the bed level is lowered over a distance Δh.

For the mathematical solution of the problem reference is made to the original publication (de Vries 1975). See also Janssen (1979, p. 123)

Fig. 3.5. Definition sketch.

If the x-axis is taken along the original bed level upstream from the mouth (x = 0) then the bed level variation $z_b(x,t)$ is described by

$$z_b(x,t) = - \Delta h \ \text{erfc} \left[\frac{x}{2\sqrt{Kt}} \right] \qquad (3\text{-}32)$$

in which the '*complementary error-function*' is described by

$$\text{erfc } y = \frac{2}{\sqrt{\pi}} \int_y^{\infty} \exp \{-u^2\}. \ du \qquad (3\text{-}33)$$

y	-1.0	-0.5	-0.2	-0.1	0	0.1	0.2	0.5	1.0	2.0
erfc y	1.84	1.52	1.22	1.11	1.00	0.89	0.78	0.48	0.16	0.005

Table 3.1 Complementary error-function.

In the first place it will be assumed that the discharge is constant. This facilities understanding. The solution of Eq. (3-32) can be used to answer the following question:

If a station $x = L_m$ is selected, at what time $t = T_m$ will the degradation have reached 50% of the final value?

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From Eq. (3-32) is follows with Table 3.1

$$\mathrm{erfc}\left[\frac{L_m}{2\sqrt{KT_m}}\right] = \tfrac{1}{2} \quad \text{or} \quad L_m \approx \sqrt{KT_m} \qquad (3\text{-}34)$$

Hence the parameter K plays a key role in the description of this morphological process.

3).

In practice the river discharge will vary in time. It can be shown (de Vries, 1975) that for K = K (t) the solution of the problem is given by

$$z(x,T) = -\Delta h \ \mathrm{erfc} \frac{x}{2\sqrt{\int^{T} K(t)dt}} \qquad (3\text{-}35)$$

Hence the 50% degradation is reached at $t = T_m$ for $x = L_m$ if

$$L_m = \sqrt{\int^{T_m} K(t)dt} \qquad (3\text{-}36)$$

Using Eq. (3-27) one may define the parameter Y with

$$Y = \int_0^{1\ \text{year}} K(t)dt = \frac{1}{3}\frac{n}{Bi}\int_0^{1\ \text{year}} S(t)dt \qquad (3\text{-}37)$$

The integral of Eq. (3-37) denotes the average yearly transport of the river.

Hence at $x = L_m$ the 50% degradation is reached after N_m years with

$$N_m = \frac{L^2_m}{Y} \qquad (3\text{-}38)$$

For a number of rivers Table 3.2 gives the value of N_m. For L_m the value L_m = 200 km has been chosen to fullfil the requirements for the parabolic model $A > 2$ to 3.

RIVER	STATION (approx. distance to sea)	D mm	i $*10^{-4}$	3a/i km	N_m centuries
Rhine (Netherlands)	Zaltbommel (100 km)	2	1.2	100	20
Magdalena (Colombia	Puerto Berrío (730 km)	0.33	5	30	2
Danube (Hungary)	Dunaremete (1826 km)	2	3.5	40	10
	Nagymaros (1695 km)	0.35	0.8	180	1.5
	Dunaujvaros (1581 km)	0.35	0.8	180	1.5
	Baja (1480 km)	0.26	0.7	210	0.6
Tana (Kenya)	Bura (230 km)	0.32	3.5	50	2
Apure (Venezuela)	San Fernando	0.35	0.7	200	4.4
Mekong (Thailand)	Pa Mong	0.32	1.1	270	1.3
Serang (Indonesia)	Godong	0.25	2.5	50	2.0
Rufiji (Tanzania)	Stiegler's Gorge	0.4	3.2	20	4.0

Table 3.2. Morphological time-scale (after de Vries, 1975) for L_m = 200 km.

3.2.4. Comparing equilibrium situations

In this Sub-Section for a number of standard problems the steady state solutions due to morphological changes will be discussed. An equilibrium situation in a river is changed into a new equilibrium. This means that the basic equations are considered for $\partial z_b/\partial t = 0$. In the most simple cases with a constant width it means that also $\partial s/\partial x = 0$ and $\partial a/\partial x = 0$.

Hence the analysis can be carried out with

$$Q = C\,B\,a^{3/2}\,i_b^{1/2} \tag{3-39}$$

and

$$S = B\,m\,u^n \tag{3-40}$$

In this elementary analysis m and n are supposed to be constant. The same holds for C or if the Strickler formula with $C = 25(a/k_N)^{1/6}$ is used the value of k_N is supposed to be constant.

The examples given below are essential for the understanding of the time-depending morphological predictions treated in Chapter 4.

Case I: Withdrawal of water from a river

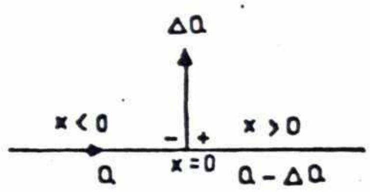


Fig. 3.6. Withdrawing water

From a river with a constant width (B) a part ΔQ of the constant discharge Q is withdrawn. What changes take place eventually (i.e. for $t \to \infty$)?

At the intake (Fig. 3.6) the width is continuous. Hence s has to be same at the upstream side (minus sign) and the downstream side (plus sign). Thus $s_- = s_+$.

Apparently

$$m\,u_-^n = m\,u_+^n \tag{3-41}$$

or

$$\frac{q}{a_-} = \frac{q - \Delta q}{a_+} \tag{3-42}$$

or with $a_+ = a_- - \Delta a$

$$\frac{\Delta a}{a_-} = \frac{\Delta q}{q} = \frac{\Delta Q}{Q} \tag{3-43}$$

The sudden change Δa at the intake has to be reflected by a step Δz_b in the bed: $\Delta z_b = \Delta a$ because the water level is continuous ($h_- = h_+$).

In this case the smaller discharge can only carry the original transport if the downstream bedslope increases eventually. With the assumption $C_- = C_+$ it follows

$$q - \Delta q = C(a - \Delta a)^{3/2} (i + \Delta i)^{\frac{1}{2}} \tag{3-44}$$

in which i is the original bedslope. Equations (3-39) and (3-44) give for the relative change of the slope

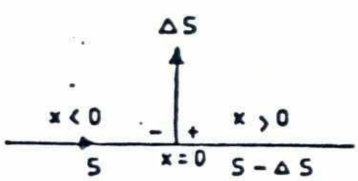
$$\frac{\Delta i}{i} = \frac{\Delta Q/Q}{1 - \Delta Q/Q} \approx \frac{\Delta Q}{Q} \quad \text{if } \Delta Q \ll Q \tag{3-45}$$

If not the assumption $C_- = C_+$ but $k_{N-} = k_{N+}$ is made then the result is

$$\frac{\Delta i}{i} = \frac{1}{\left[1 - (\Delta Q/Q)\right]^{4/3}} - 1 \tag{3-46}$$

The proof is left to the reader.

Case II: Withdrawing sediment from a river



From a river with a constant width B and a constant discharge Q, sediment is withdrawn at a constant rate ΔS from t = 0. The sediment is used for building purposes.

Fig. 3.7. Withdrawing sediment

The new equilibrium situation ($t \to \infty$) can be estimated as follows. Using again $s = m\,u^n$ it follows

$$Q = B.a.u = B.a.(s/m)^{n^{-1}} \qquad (3\text{-}47)$$

Hence (see Fig. 3.7):

$$a_-.s^{n^{-1}} = (a + \Delta a)(s - \Delta s)^{n^{-1}} \qquad (3\text{-}48)$$

if Δa is the increase of the depth for $x > 0$.
From Eq. (3-48) follows with $a = a_-$

$$\frac{\Delta a}{a} = \left[\frac{1}{1-(\Delta S/S)}\right]^{n^{-1}} - 1 \qquad (3\text{-}49)$$

In the downstream reach the slope has to decrease. This can be estimated by means of Chézy formula and the assumption $C_- = C_+$.

From $Q_- = Q_+$ and $B_- = B_+$ follows

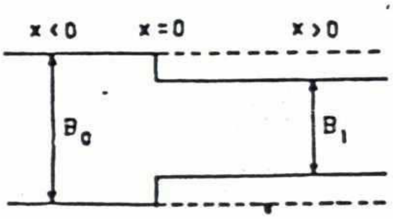
$$a_-^{3/2}\, i_-^{1/2} = (a_- + \Delta a)^{3/2}\,(i_- - \Delta i)^{1/2} \qquad (3\text{-}50)$$

thus

$$\frac{\Delta i}{i} = 1 - \left[1 - (\Delta S/S)\right]^{3/n} \qquad (3\text{-}51)$$

The exponent $3/n$ becomes $10/3n$ if $k_{N-} = k_{N+}$ is assumed instead of $C_- = C_+$.

Case III: Change of width



A river with fixed banks and constant width B_o is narrowed for $x > 0$ to a new width B_1. The discharge Q is constant. Instead of the old depth (a_o) for $x > 0$ and $t \to \infty$ the new depth becomes a_1.

Fig. 3.8. Change of width

From $S_o = S_1$ and $Q_o = Q_1$ follows with $s = m\,u^n$.

$$\frac{a_1}{a_o} = \left[\frac{B_o}{B_1}\right]^{\frac{n-1}{n}} \qquad (3\text{-}52)$$

For x = 0 there is a bottom step because the water level is continuous. The bottom step is $\Delta z_b = a_1 - a_o$.

In this case the new slope (i_1) is smaller than the old slope (i_0). With the assumption $C_o = C_1$ it follows easily from the Chézy equation

$$\frac{i_1}{i_o} = \left[\frac{B_1}{B_o}\right]^{\frac{n-3}{n}} \tag{3-53}$$

If one assumes $k_{No} = k_{N1}$ instead of $C_o = C_1$ then the exponent in Eq. (3-53) becomes: $(4n - 10)/3n$.

Remarks:

(1) For a varying discharge and a width that does not vary too much the above given analysis can also be given, using $f(Q)$. This is the analysis given in Section 2.5 resulting in Eq. (2-8). The estimate for the new slope according to Eq. (3-53) therefore also holds for a varying discharge. This equation is graphically represented in Fig. 3.9.

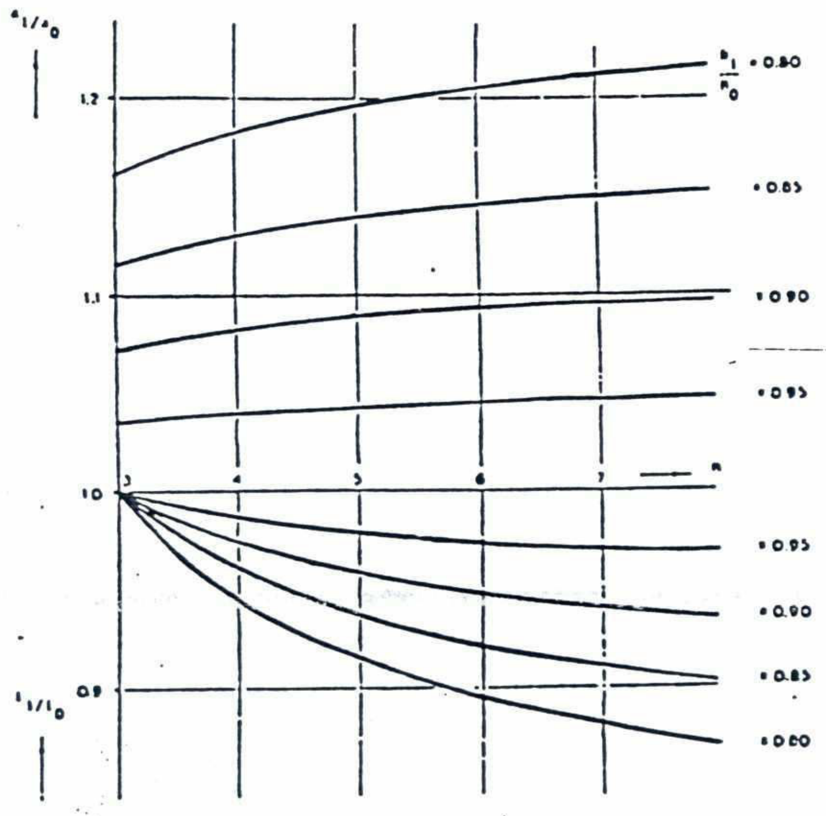


Fig. 3.9. Consequences of constriction of river width.

(ii) According to Eq. (3-53) as also can be seen from Fig. 3.9 the new slope is equal to the old slope if n = 3. According to the Meyer-Peter and Mueller (1948) formula this is the case for a large transport of coarse bed material. It should be remarked that the above given analysis is an elementary one. In a practical problem a more in-depth-approach is advised taking into account the change in roughness that may occur.

(iii) In the three cases described above a discontinuity in Q, S and B respectively lead at x = o to a discontinuity in the depth. As the water level is continuous (no hydraulic jump) this will result in a step Δz_b in the bed. The bed *level* has then not yet been obtained. This is because the bed level is governed by the water level (boundary condition) at the downstream end of the channel. The bottom*level* Δz_b is reached only for $t \to \infty$.

3.2.5. Influence of suspended load transport

So far the morphological processes have been described by using a transport formula s = f(u), thus assuming that the sediment transport can be described by the *local* hydraulic conditions. This is true when *bedload transport* is present. In the case of *suspended load transport*, however, it is only true for steady uniform flow. This also the situation for which transport formulae are developed and tested experimentally.

The first question is how transport can be estimated for *non-uniform flow*. It is customary and justified to introduce the friction term in the differential equation for the backwater curve (equation of Bélanger) as if the flow were uniform. Similarly this can be done for bedload. The local shear stress can by means of the Chézy formula be transformed into the flow velocity. Hence $ai = u^2/C^2$.

For non-uniform flow with suspended load this cannot be done without restrictions. This is related to question what *local* means in this respect. For non-uniform flow and suspended load a distinction has to be made between *transport* and *transport capacity*:

transport (s') = $\int^a u\phi \, dz$

transport capacity s = f(u) i.e. following from a formula

in which ϕ denotes the sediment concentration.
Generally in the case of suspended load

- $\partial u/\partial x > o$ gives $s' < s$
- $\partial u/\partial x < o$ gives $s' > s$

The explanation can be found from the following rather extreme example. Suppose a steady uniform flow in a laboratory flume with a fixed bed for $x \leq 0$ and a mobile bed of fine sediment for $x > 0$. The mean concentration over the vertical is $\overline{\phi}$. For large values of x the average equilibrium concentration $\overline{\phi}_c$ is reached. It requires a certain *adaption length* to reach the equilibrium concentration over the entire depth. This equilibrium concentration belongs to the flow conditions present and the sediment characteristics.

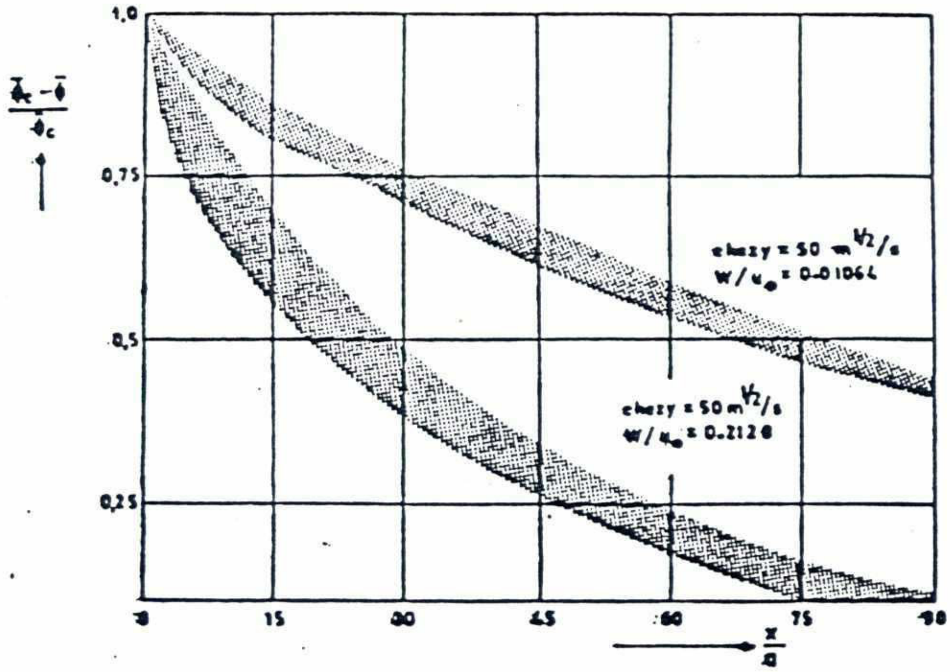


Fig. 3.10. Adaptation concentration vertical (after Galappatti, 1983)

In Fig. 3.10 the theoretical adaptation of the concentrations determined by Galappatti (1983) is given. It has been assumed that at a distance z = 0.0125 a the concentration is instantaneously equal to the equilibrium concentration. (here for $x > 0$).

It appears that the *adaptation length* depends on the parameter W/u_* and the roughness (C-value). In this example no sediment supply at x=0 is present. Hence erosion will follow for x > 0. In Fig. 3.10 the situation is considered for which no erosion has yet taken place (thus small value of t).

In practice morphological computations are carried out with numerical models with discrete values Δx and Δt. If L is the adaptation length than two cases can be distinguished.

- $\Delta x > L$. In this case the one-dimensional morphological model can be used for suspended load as for bed load.
- $\Delta x < L$. Now in fact a two dimensional (vertical) model has to be used. The concentration $\phi(x,z,t)$ has to be calculated and the transport $s' = \int u \phi \, dz$ has to be determined before bed-level variations by means of the equation of continuity for the sediment can be computed.

Between these two cases Galappatti (1983) has developed an asymptotic approach which extends the region for which a one-dimensional approach can be used. (See Sub-Section 3.3).

3.3. Two-dimensional approaches

3.3.1. Flow in river bends

To understand the bed level in river bends some attention has to be paid to the details of the flow in bends of open channels.

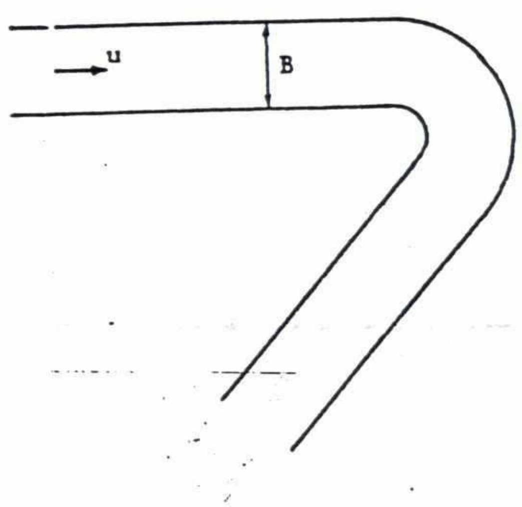


Fig. 3.11. Circular bend.

In Fig. 3.11 a circular bend in a laboratory flume is sketched. The upstream and downstream parts of the flume are straight. The (fixed) bed is horizontal.

In the first approach friction is neglected. This implies that potential flow can be postulated. In this problem a natural coordinate system is appropriate. Here s is the coordinate along the streamline and n the direction normal to the flow line (thus s-n is the horizontal plane). The b-axis is perpendicular to the s-n plane. The components of the velocity vector are u_s, u_n and u_b respectively. According to the definitions of s,n and b it holds $u_s \neq 0$; $u_n = 0$ and $u_b = 0$. However, derivatives of u_n and u_b exist.

For steady flow the momentum equations in the natural coordinate system read:

$$u_s \frac{\partial u_s}{\partial s} = -\frac{1}{\rho}\frac{\partial p}{\partial s} + a'_s \qquad (3\text{-}54)$$

$$\frac{u_s^2}{r} = -\frac{1}{\rho}\frac{\partial p}{\partial n} + a'_n \qquad (3\text{-}55)$$

$$0 = -\frac{1}{\rho}\frac{\partial p}{\partial b} + a'_b \qquad (3\text{-}56)$$

in which p is the pressure. The right-hand sides contain the components of the acceleration $\vec{a}'$ that is present in addition (e.g. due to gravity and/or friction). In Eq. (3-55) the radius of curvature of the streamline is indicated by the parameter r.

If friction is neglected only g is present in $\vec{a}'$. This implies $a'_s = 0$; $a'_n = 0$ and $a'_b = g$. Hence Eq. (3-56) indicates that in the (vertical) b-direction the *hydrostatic pressure distribution* is present.

If the piezometric head is measured from the bed level, then the Bernoulli equation along the streamline gives

$$\frac{u_s^2}{2g} + a = \text{constant} \qquad (3\text{-}57)$$

The absence of friction means that potential flow can be postulated. This means that here is one Bernoulli-constant for the entire flow field.

In the n-direction holds

$$\frac{u_s^2}{r} = -\frac{\partial}{\partial n}\{g(a-z)\} \qquad (3\text{-}58)$$

if z is the distance to the bottom.
Hence

$$\frac{u_s^2}{gr} = -\frac{\partial a}{\partial n} \qquad (3\text{-}59)$$

Differentiation of Eq. (3-57) with respect to r gives

$$\frac{u_s}{g}\frac{\partial u_s}{\partial r} + \frac{\partial a}{\partial r} = 0 \qquad (3\text{-}60)$$

The coordinate n is taken positive in the direction of the centre of curvature. Hence $\partial/\partial r = -\partial/\partial n$. Combining Eqs. (3-59) and (3-60) gives

$$\frac{u_s^2}{gr} = -\frac{\partial a}{\partial n} = \frac{\partial a}{\partial r} = -\frac{u_s}{g}\frac{\partial u_s}{\partial r} \qquad (3\text{-}61)$$

or

$$u_s = r^{-1} \qquad (3\text{-}62)$$

Hence the assumption of potential flow leads to the conclusion that the largest flow velocities are found in the inner bend.

Now the influence of friction can be taken into consideration. In a vertical there is only one value of da/dr. As u_s is due to friction larger at the water surface than near the bed it holds

- water surface $\frac{u_s}{g}\frac{du_s}{dr} > \frac{da}{dr}$
- bed level $\frac{u_s}{g}\frac{du_s}{dr} < \frac{da}{dr}$

Hence the water particles near the water surface are slightly deviating outward whereas the water particles near the bed are slightly deviating inward ($u_s g^{-1}\, du_s/dr$ relatively small). Thus in the bend there exists a *helicoidal flow*. This helicoidal flow is composed of a main current in the direction of the channel axis and a circulation in the cross-section (Fig. 3.12).

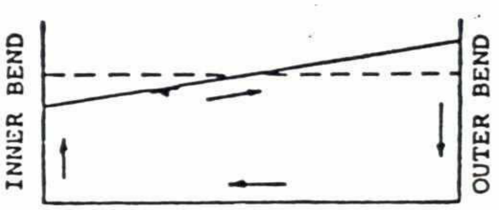


Fig. 3.12. Circulation in the cross-section

The water level in the bend has a slope perpendicular to the river axis. The value of the difference in head (ΔH) accross the river can be estimated using Eq. (3-59).
If r_o is the radius of curvature of the inner bend then

$$\Delta H = \int_{r_o}^{r_o+B} \frac{u_s^2}{gr} \, dr \tag{3-63}$$

Example:
For the River Waal at a certain place $r_o = 2$ km and $B = 260$ m. The average flow velocity is $\bar{u}_s = 1.2$ m/s. The Eq. (3-63) can be approached by

$$\Delta H \approx \frac{\bar{u}_s^2}{g(r_o + \frac{1}{2}B)} \cdot B = \frac{(1.2)^2 \cdot 260}{9.8(2000 + 130)} = 0.02 \text{ m} \tag{3-64}$$

So far it has been assumed that the bed level in the bend is horizontal. However, when the bed is mobile, this is not a stable situation. Due to the helicoidal flow near the bed the transport will have a direction to the inner bend. The means that the transport vector has a component in the direction transverse to the channel axis. The inner bend becomes shallower while the outer bend gets a greater depth. The cross slope becomes so steep that the perpendicular transport component is compensated by a transport downwards due to gravity. The change of the bed level implies that in a natural river Eq. (3-62) will not hold anymore. The velocity u_s is relatively large then in the *outer* bend.

By Van Bendegom (1947) and Rosovskii (1957) the magnitude of the secondary velocity has been studied theoretically assuming for the velocity distribution in the vertical a power law and a logarithmic law respectively. More details can be found in Jansen (1959, p. 59).

In Fig. 3.13 the two theoretical profiles are compared with measurements.

It has to be noted that these measurements are not easy to be carried out accurately. This is because the velocity vector $\vec{u}$ has the component u in the direction of the river axis and the radial component v. Generally $u \gg v$. This means that a small error in the measurement of u leads to a large error in v.

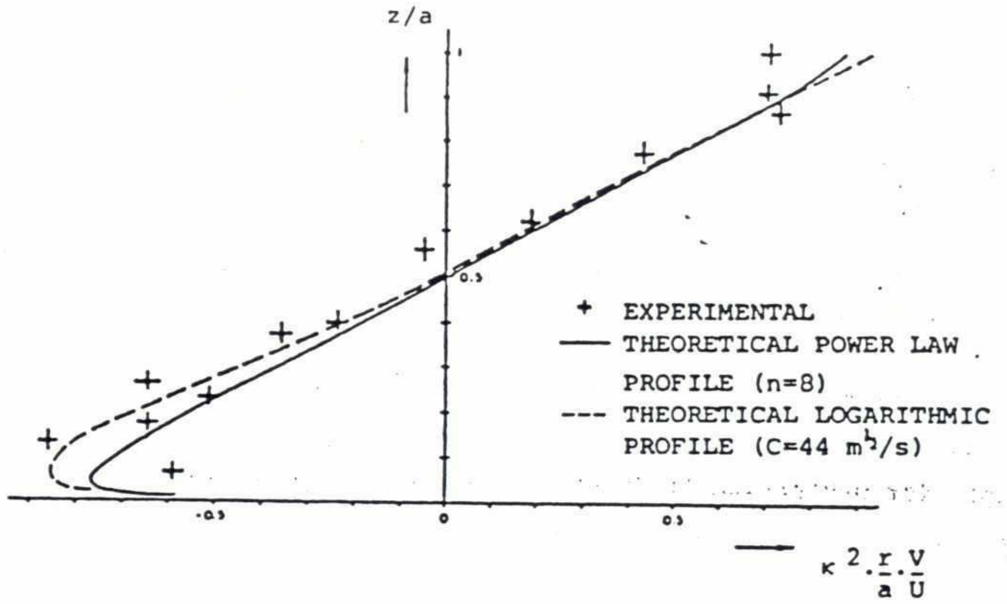


Fig. 3.13. Secondary current (v) in a riverbend: comparison of theory and measurement (after Kondrat'ev *et al*, 1959)

As $v << u$, the total shear stress at the bed makes only a small angle δ with the river axis (order of magnitude $\delta = 1$ to $2°$).

The flow through bends in open channels will not be discussed here in detail. For a thorough investigation on this topic reference is made to de Vriend (1981).

The studies by Van Bendegom (1947) and Rosovskii (1959) assume that locally the velocity field is adapted to the local radius of curvature. According to de Vriend and Struiksma (1983) this can only be the case if there is much friction! In general there will be phase lag. The velocity field lags behind the change in geometry (expressed e.g. in the radius of curvature). It seems that this lag is essential in explaining the bed topography in river bends.

Experimentally it has been shown (de Vries, 1961) that in the branches of the Rhine in the Netherlands phase lags do exist in the bends between some morphological parameters. If along a line parallel to the river axis the flow velocity $u(s,r)$, the depth $a(s,r)$ and the mean grain size $D_m(s,r)$ are measured, then these morphological parameters vary in the s-direction. By treating the experimental data statistically it was shown that u reacts more downstream than a on a variation in the radius of curvature and D_m reacts more downstream than u. These tendencies were also found in a scale model of a reach of the Lower Rhine. This demonstrates the complexity of the flow in a curved alluvial channel especially if the bed material is not uniform, as usually is the case in nature.

3.3.2. Bed configurations in bends

The bed configuration in river bends are of paramount importance in river engineering. In the bends the depth at the outerbend is of importance in the design of bankprotections. Also the availabe width in the bend with a certain required depth for navigation is important to know. Between the bends i.e. at the *river crossings* the available depth is of importance to navigation.

An early statistic research was carried out by Lely (1922) for the Rhine branches in the Netherlands. The research was carried out for rivers reaches with constant width (B) and fixed banks. Lely's conclusions were

- The mean depth across the river in the bends is about the same as at the crossings.
- The change in the curvature (r) of the bend leads to a change in the transverse slope (β) of the bed. The transverse slope reacts about 1½ B downstream of a change in r (phase lag).
- The magnitude of β depends on r with (metric units):

$$\beta = \frac{22}{r} \qquad (3\text{-}65)$$

Based on the work of van Bendegom (1947) in NEDECO (1959) a method is given to compute for a hypothetical river with fixed banks the depth accross the river for an infinitely long circular bend. Hence this approach does not consider a variation in the s-direction. Using the expression for the bed shear stress it was derived:

$$\frac{da}{a^2} = \frac{3}{2}\alpha\, i_x \frac{dr}{r\Delta D} \qquad (3\text{-}66)$$

in which

r = radius of curvature of the circular bend

D = mean grain size

i_x = longitudinal slope

and

$$\alpha = \frac{2n^2}{\kappa^2 (n+2)(n+3)} \qquad (3\text{-}67)$$

The parameter n is here the exponent of the powerlaw for the vertical velocity distribution ($u(z) = z^{n-1}$)

The additional hypothesis

$$i_x \cdot r = i_o \cdot r_o \qquad (3\text{-}68)$$

(in which the subscript o stands for the outer bend) is questionable. Equations (3-66) and (3-68) lead to

$$\frac{1}{a} - \frac{1}{a_o} = \left\{ \frac{1}{r} - \frac{1}{r_o} \right\} \frac{1.5\alpha i_o r_o}{\Delta D} \qquad (3\text{-}69)$$

This equation sometimes gives good results especially as the coefficient can be used to tune the equation, this in fact also holds for D.

Apmann (1972) studies the same problem. He argues that

$$\frac{da}{dr} = m \frac{a}{r} \qquad (3\text{-}70)$$

in which the coefficient m depends on the *flow parameter* $\Delta D/ai$. The maximum depth a_{max} in the outer bend follows then from

$$\frac{a_{max}}{a} = \frac{(m+1)(1-r_i/r_o)}{1 - (r_i/r_o)^{m+1}} \qquad (3\text{-}71)$$

From measurements of the Buffalo Creek a value m = 2.5 is deduced by Apmann. For the Rhine branches in the Netherlands m = 7 is found.

Odgaard (1981) concludes that for this axial-symmetrical approach the expression.

$$\left[\frac{da/dr}{a/r} \right]_{axis} = K \qquad (3\text{-}72)$$

is used by various authors albeit that the expression for K differs. According to Odgaard the expression $K = 2F_D^2$ is used by van Bendegom (1947) with $F_D = \overline{u}/\sqrt{g\Delta D}$. Odgaard uses laboratory measurements from Zimmerman & Kennedy (1978) and his own data from the Sacramento River. He concludes that his expression for K has to be preferred above the ones used by others. Odgaard does not discuss the work of Apmann (1972).

The above indicated approaches all considered axial-symmetrical flow and therefore neglect phase lags between u and a. Moreover uniform bed material is postulated (no grain sorting in the bends).

A completely different approach to predict the bed topography is river bends is indicated by Einstein. This is a statistical analysis and synthesis based on the following assumptions. The basic idea is that for a river reach with (nearly) constant width

- The regime Q(t) is the same.
- The composition of the bed material is the same.
- The influences of the banks can be neglected if B/a >> 1.

For each cross-section therefore only the *geometry* in the horizontal plan is different. The geometry is expressed by the radius of curvature only.

For the river considered part of the reach is used to deduce the statistical parameters and they are used to predict the bed morphology downstream. Therefore measured bed level $z_b(y)$ are matched with a linear series of (orthogonal) legendre polynomials $P_r(y)$. Here y is measured perpendicular to the river axis.

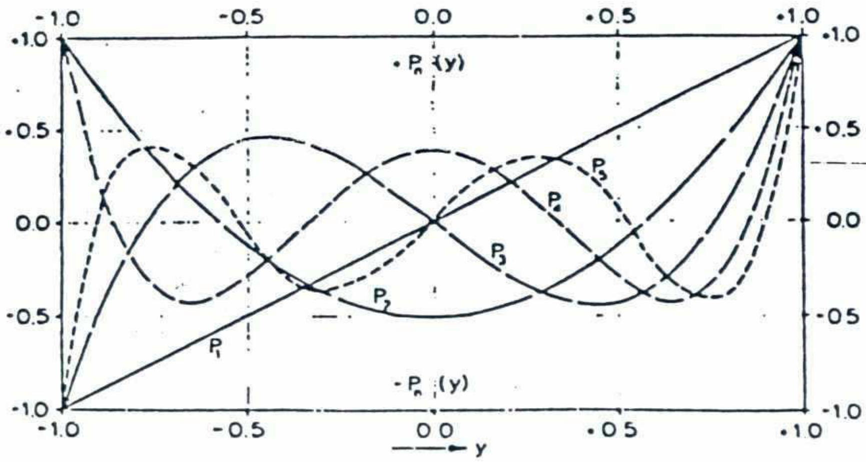


Fig. 3.14. Legendre polynomials

The functions $P_r(y)$ are given in Fig. 3.14.

$$z_b(y) = a_o P_o(y) + a_1 P_1(y) + \ldots\ldots + a_r P_r(y) + \ldots\ldots + a_N P_N(y) \qquad (3\text{-}73)$$

The parameters a_r for any cross-section n are related to the curvature C of the cross-section p upstream of n according to

$$a_{r,n} = A_{o,r} + \sum_{p=1}^{p_o} A_{p,r} \cdot C_{n-p+1} \qquad (3\text{-}74)$$

The parameters a_r belong to a certain cross-section whereas the parameters A belong to the entire river.

The procedure is now as follows:

Analysis:

(i) Use the measured bed levels $z_b(y)$ to determine the coefficient a_r from Eq. (3-73).

(ii) For the river with known geometry (C) determine the parameter A from Eq. (3-74).

Synthesis:

(i) In Eq. (3-74) the parameters A are now known. For another part of the river C is known thus $a_{r,n}$ can be computed from Eq. (3-74).

(ii) With known parameters $a_{r,n}$ in Eq. (3-73) the bed level $z_b(y)$ can be computed.

It remains to be seen how many (N) polynomials have to be used. Moreover the number (p_o) of cross-sections upstream of the cross-sections considered has to be selected.
Einstein (1971) used in his application to the Missouri River N = 6 and he took p_o = 15 which means a distance of about 2 km.

Nijdam (1973) applied the method to the Waal River. He also took N = 6 but selected p_o = 30 which means that the geometry upto a distance of 3½ km upstream of the cross-section considered is responsible for the bed level in that cross-section.

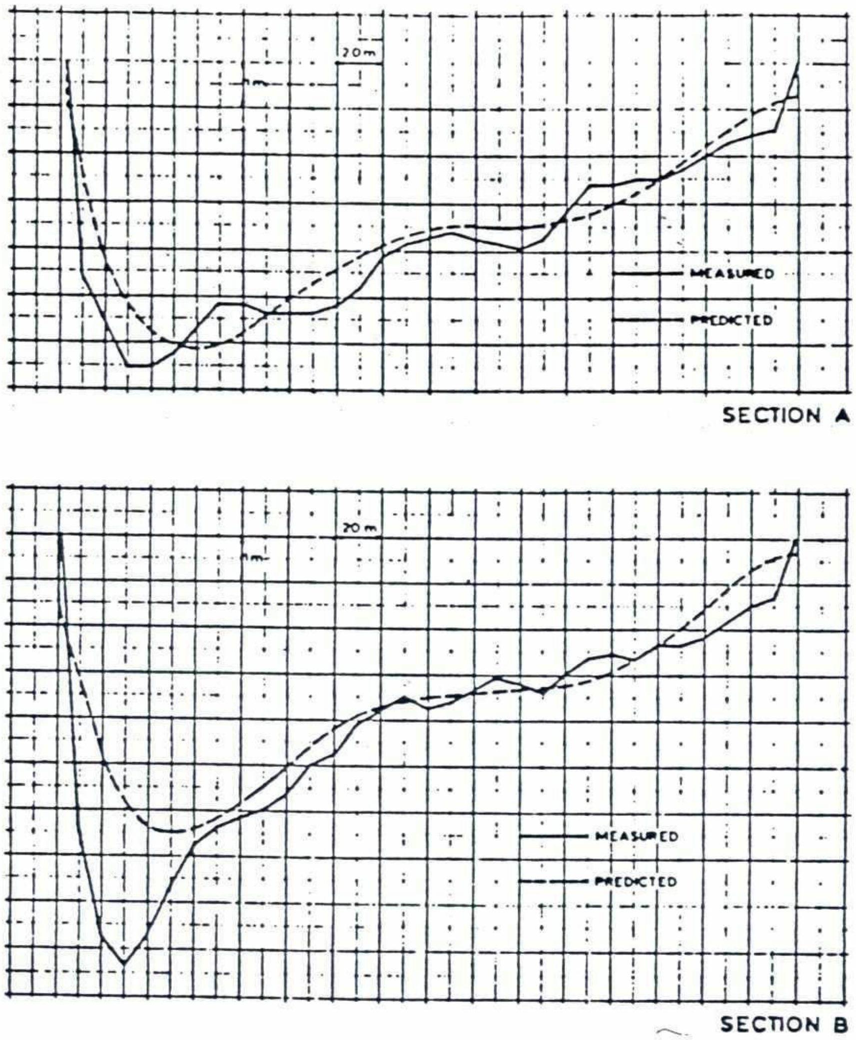


Fig. 3.15. Statistical prediction of bed levels (Waal River after Nijdam, 1973)

Some results are given in Fig. 3.16. It has to be remarked that the result is sensitive to the selection of p_o. Moreover Nijdam reports that the results are less good at the crossings. With respect to Fig. 3.16 it has to be remarked that the disagreement for section B between measured and predicted bed level at the left hand bank seems to be due to the fact that locally a groin is present. Hence the assumption of absence of wall-influences is there not justified.

The statistical method treated have has another essential underlying assumption *viz* the presence of an *alluvial* bed. If part of the bed is not erodible (rock, clay-layers, armoured areas) then the prediction fails. Obviously the method also fails in the case for the Meuse River near Roermond in the Netherlands. The bed consists there of gravel that only is transported at large discharges. At moderate discharges some sand is transported over the gravel bed. The bed profile of the bends in this river differs from an alluvial river (Fig. 3.16). There is no sand deposition in the outer bend. Via the crossing the sand is transported to the inner bend. There sand deposits are present. In this river morphological predictions are difficult, because transport formulae do not apply. Hence the transport of sand in a reach is governed by the *supply* at the upstream end. This is a similar situation as can be present in an alluvial river with suspended load. For this part of the Meuse River the sediment transport is *smaller* than the sediment transport capacity. The gravel layers prevent erosion.

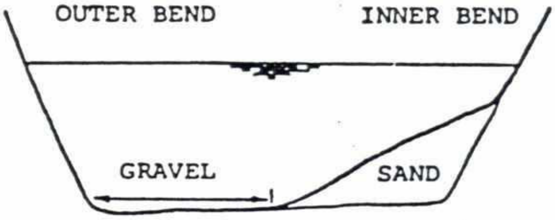


Fig. 3.16. Cross-section Meuse River near Roermond

Gradually it becomes possible to compute bed levels in alluvial bends based on the hydrodynamic equations. In the two-dimensional (horizontal) water equations the effect of the helicoidal flow has to be incorporated. The sediment equations have also to be taken for the two-dimensional case. Moreover some formula has to be adopted for the direction of the transport.

In Fig. 3.17 some results are given of the two-dimensional model SEDIBO being developed by the Delft Hydraulics Laboratory (Schilperoort *et al*, 1984). The results look promissing in spite of the fact that the presence of uniform sediment was assumed. The computations have been carried out for a constant discharge.

The development of these types of morphological models is in the direction of also including the case of a sediment mixture and/or the presence of suspended load.

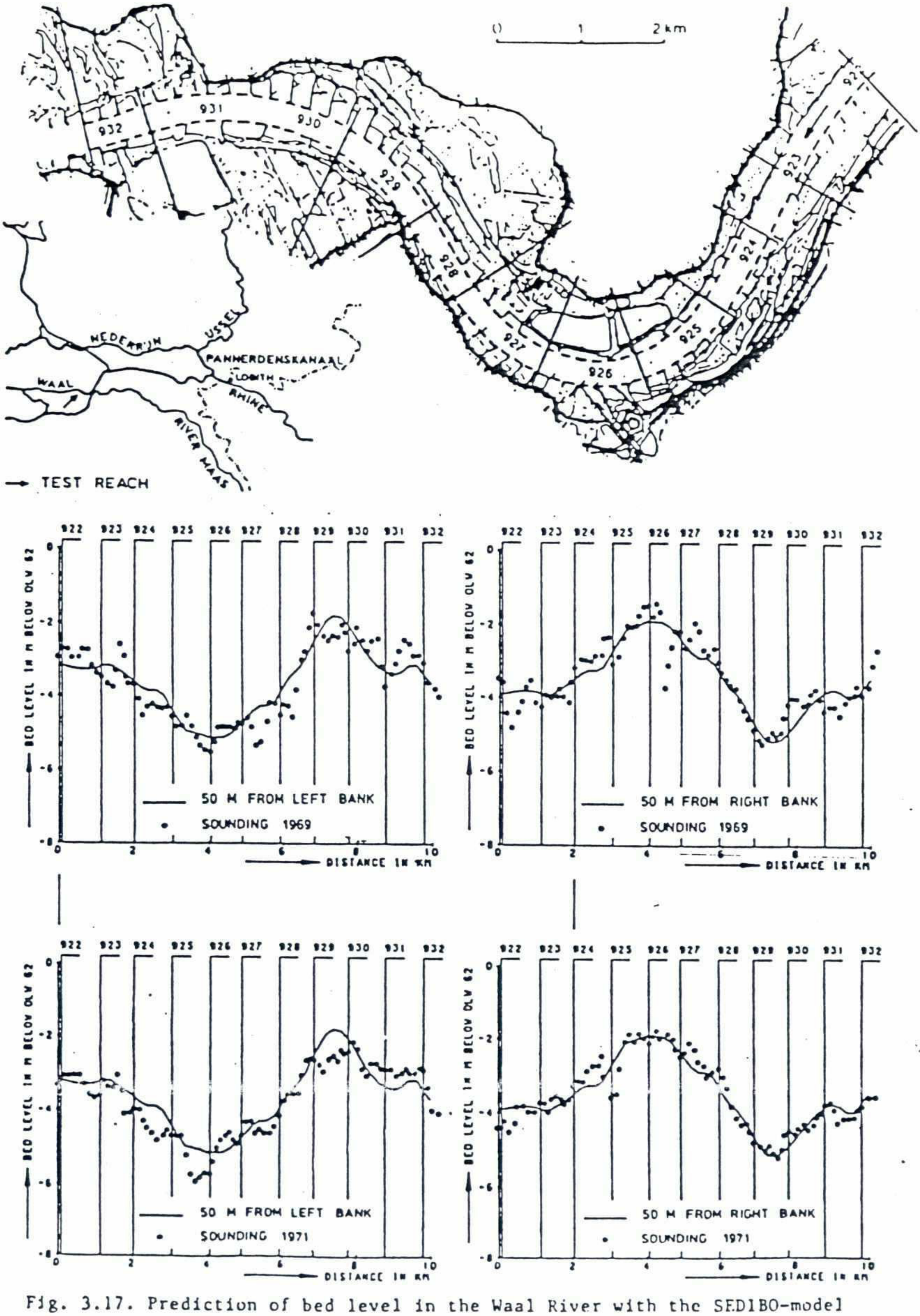


Fig. 3.17. Prediction of bed level in the Waal River with the SEDIBO-model (after Schilperoort *et al*, 1984)

3.3.3 Two-dimensional vertical

In Sub-Section 3.2.5 it has already been indicated that in the case of suspended load changes in the geometry over relatively short distances may cause that the local sediment transport is not equal to the local sediment transport capacity.

The mathematical models with one space dimension can then not be used. In stead a two-dimensional model (vertical) is applied to determine the sediment concentration $\phi(x,z)$. In that case no sediment transport formula is applied but the transport is determined by the integral $\int\phi(z)\ u(z)\ dz$ over the depth.

For a first introduction in this type of models reference can be made to Kerssens *et al*(1979).
The concentration (x,z) is determined by solving the two-dimensional convection-diffusion equation

$$u\frac{\partial\phi}{\partial x} - \frac{\partial}{\partial z}\{ W\phi + \varepsilon\frac{\partial\phi}{\partial z}\} = 0 \qquad (3\text{-}75)$$

in which

W = fall velocity
$\varepsilon = \varepsilon(z)$ = eddy viscosity

Note that if the flow is steady uniform for both water and sediment, than the first term of Eq. (3-75) disappears. Equation (3-75) then leads to the well-known Rouse-distribution for uniform flow. The solution of Eq. (3-75) requires a boundary condition at the bed. The discussion of these type of models is outside the scope of these lecture notes.

4. Morphological predictions

4.1. General

Artificial interference in a river system by engineering works (discharge regulation, water level regulation, normalization, canalization etc.) will lead to a response of the river. It is required to predict the response to the artificial interference. However, only part of the response can be predicted.

For instance if the discharge of a river is controled by installing a large dam (like in the River Nile and the Zambezi River) it can be expected that major changes can take place downstream in the characteristics of the river. To the writer's knowledge no prediction techniques have been successful in the respect. At best the *regime theory* can be used to predict some tendencies. This 'theory' (see Graf, 1971, p. 243-272) has little to do with a theory in the usual sense. River characteristics are related statistically. However, to use this statistical relations to predict changes when the boundary conditions (e.g. by discharge control) are altered is questionable. No rivers have been analysed thoroughly in this respect. This also due to the fact that changing of the appearance of a river takes time and the regime 'theory' does not contain time as a parameter.

The best predictions of changes can be carried out when the morphological problems involved can be described in a deterministic way. This is why in this Chapter the predictions are restriced to cases in which the banks are fixed or when the mobility of the banks is much smaller than that of the (alluvial) bed.

The morphological predictions can in principle be obtained by scale models and mathematical models. However, some problems are too complicated to give a reliable prediction.

As an example the morphological problems present at the access to the ports of Brazzaville (Congo) can be mentioned. Figure 4.1 gives the situation of the 'Port à Grumes' (Timberport) along the Congolese branch of the Congo River.

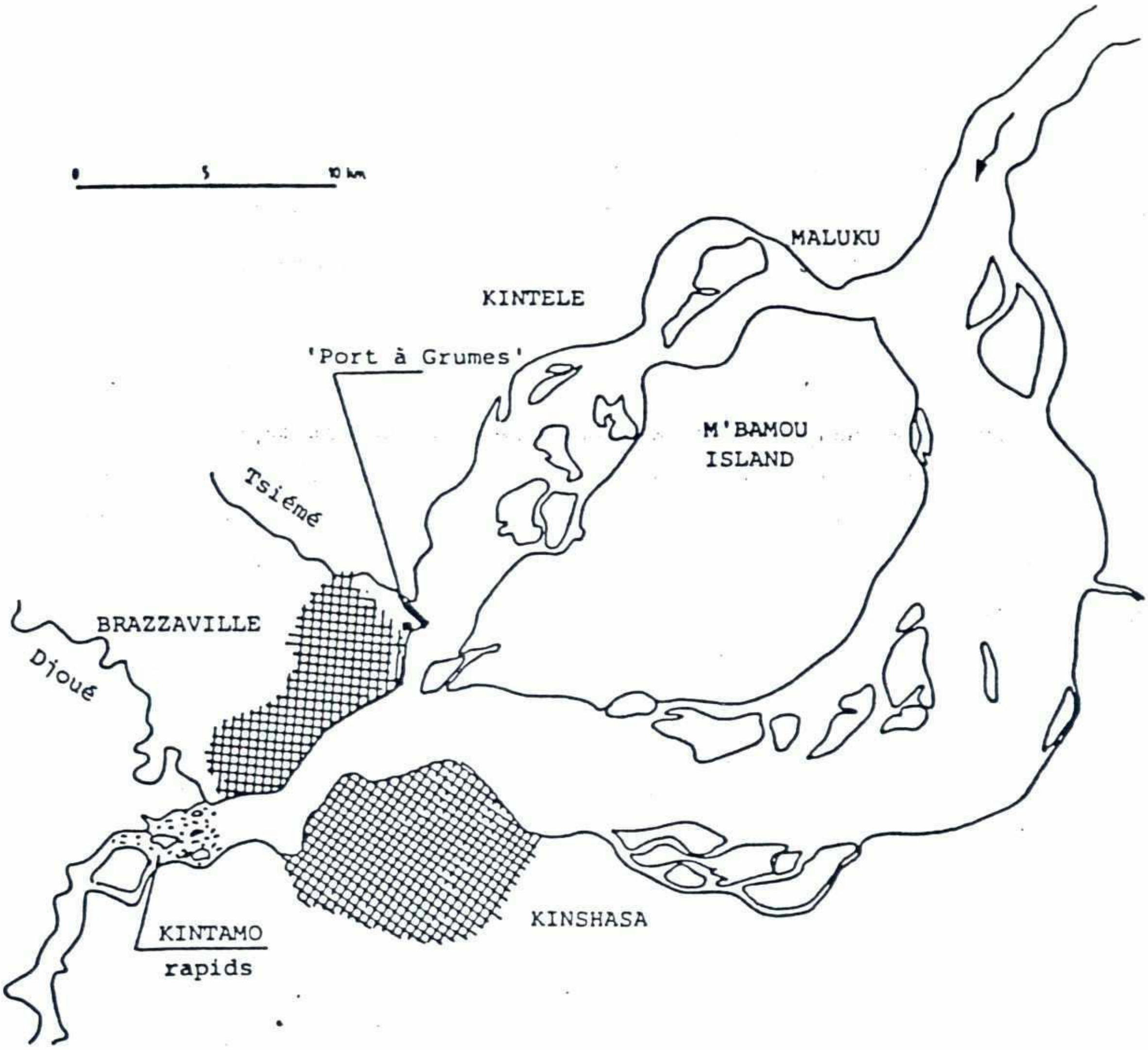


Fig. 4.1. Situation 'Port à Grumes', Brazzaville

The main branch of the river flows along the other side of the M'bamou Island. (Zairese branch). Just downstream of Brazzaville and Kinshasa the Kintamo rapids are situated. This is why at the port the logs transported along the river are loaded on trains to be transported to the coast (Pointe Noire). The Congolese branch is characterized by islands and bars. Although the sediment transport of the Congo River is relatively small (see Table 1.1) nevertheless serious morphological problems can occur ($\overline{D}$ = ½ mm).

The 'Port à Grumes' has been designed in the sixties when the deep channel of the Congolese branch had a favourite position with respect to the river bank near Brazzaville. In 1972 the port was completed.
In 1977 the access to the port became difficult. In 1982 the deep channel of the Congolese branch was located at a large distance from the river bank. (Fig. 4.2).

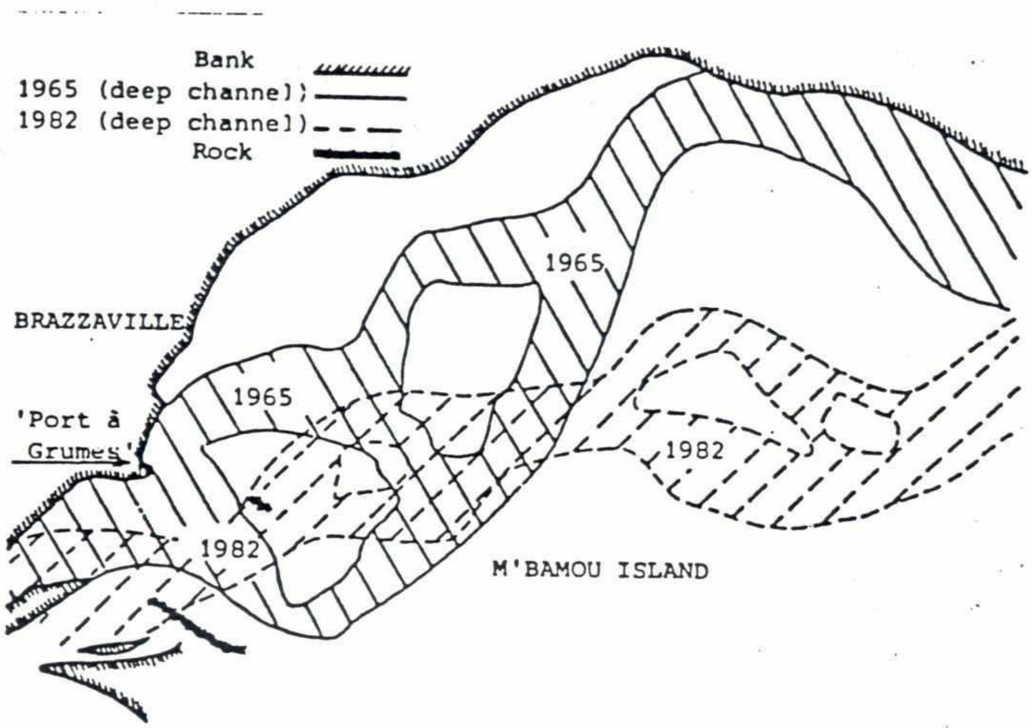


Fig. 4.2. Channel changes of the Congo River near Brazzaville

The prediction of these changes in the position of the channel(s) by means of a scale model is not possible because a large area has to be reproduced. At present (1985) also a mathematical model is not available for quantitative forecasts. This is because the flow pattern has a strong three-dimensional character governing the distribution of sediment upstream of the bars and the islands..

Given the size of the problem it is questionable whether river engineering works (perhaps except maintenance dredging) can improve the situation. Logically the problems are most serious for navigation in dry years (see also Fig. 1.2).

In the following sections the possible morphological prediction for some cases are treated. It will become clear that the river has to be schematized considerably to make morphological computations possible. The predictions are only carried out for problems that can be schematized in one space dimension.

4.2. Withdrawal of water

4.2.1. Principle

In Sub-Section 3.2.4 (Case I) the basic principle is considered on treating the the problem of withdrawal of water from a river. In practice the water is withdrawn for irrigation, industrial water, cooling water, etc.

The case is taken in which water is withdrawn at $x = L$ from the varying discharge $Q(t)$ of the river. The discharge withdrawn (Q) may also vary in time. Downstream (x L) the reaction of the river has to be considered. In this case the banks are supposed to be fixed and the bed material is uniform.

First of all the new equilibrium situation (Subscript 1) is investigated. The probability distribution $f_o\{Q\}$ of the the old situation changes into $f_1\{Q\}$ for the new situation.
As the yearly sediment transport is the same in the two cases:

$$\int_0^\infty S(Q)\,.\,f_o\{Q\}dQ = \int_0^\infty S(Q).\,f_1\{Q\}\,dQ \qquad (4-1)$$

Using as an approximation a powerlaw for the sediment transport, i.e. using Eq. (2-6) for B = constant gives

$$\frac{i_{b1}}{i_{b0}} = \left[\frac{\int_0^\infty Q^{n/3} f_0\{Q\}dQ}{\int_0^\infty Q^{n/3} f_1\{Q\}dQ}\right]^{3/n} \qquad (4-2)$$

As less water has to transport the same amount of sediment, i_b i_{bo}.
At the mouth for the same reason the depth will decrease.
For a constant width B the ratio for the depths in the mouth (a_o /a_{oo}) can be deduced from Eq. (2-10)

$$\frac{a_{01}}{a_{00}} = \left[\frac{\int_0^\infty Q^n f_1 Q\{dQ\}}{\int_0^\infty Q^n f_0 Q\{dQ\}}\right]^{1/n} \qquad (4-3)$$

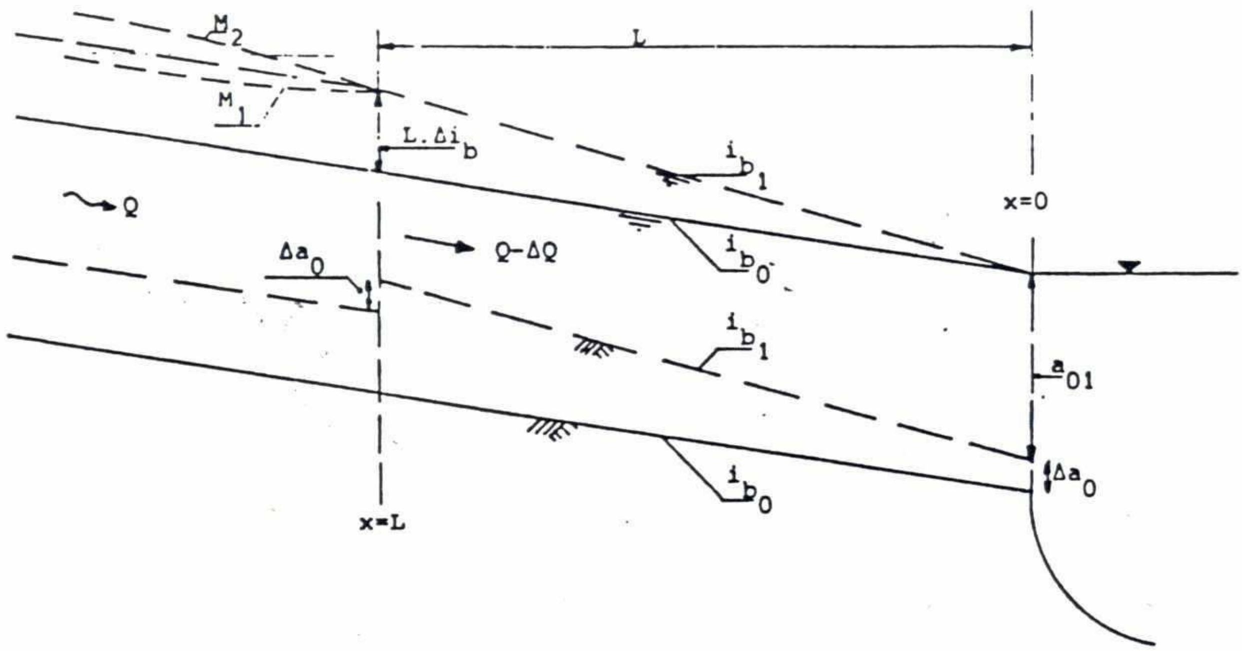


Fig. 4.3. Withdrawal of water at x = L

In Fig. 4.3 the changes of the bed are drawn for a constant discharge. The water level at the intake rises with an amount $L \cdot \Delta i_b$. At the mouth the depth reduces with $\Delta a_o = a_{oo} - a_{o1}$.
Note that the same depth reduction is present at the intake!
Just downstream of the intake the bed level rises eventually with $\Delta a_o + L \cdot i_{b1}$

Remark:
It requires some additional analysis to see how Fig. 4.3 would look like for a regime with $f_o(Q)$ which is transformed into $f_1(Q)$ due to the withdrawal of water. The bed levels remain the same but the water levels have to be given further attention.

Downstream of the intake Fig. 4.3 can be interpreted as the situation for a discharge Q_{d1} for which just uniform flow exists with a normal depth just equal to a_1. This discharge is found from

$$a_0^{-n} \int_0^{\infty} Q^n f_1(Q) dQ = a_1^{-n} \cdot Q_{d_1}^n \qquad (4\text{-}4)$$

Upstream of the intake uniform flow with normal depth equal to a is present for a discharge Q_{do} with

$$a_o^{-n} \int_0^{\infty} Q^n f_0(Q)\, dQ = a_o^{-n} Q_{do}^{n} \qquad (4\text{-}5)$$

The continuity of *sediment* in the new equilibrium situation requires

$$a_1^{-n} Q_{d1}^{n} = a_o^{-n} Q_{do}^{n} \qquad (4\text{-}6)$$

As $a_1 < a_o$ it follows $Q_{d1} < Q_{do}$. The character of the backwater curve for the steady flow in Fig. 4.3 upstream of the intake depends now in the value of ΔQ (the discharge withdrawn). Upstream of the intake the flow is uniform if just $\Delta Q = Q_{do} - Q_{d1}$. Other possibilities are:

(i) $\Delta Q > Q_{do} - Q_{d1}$. The upstream discharge is now $Q_o = Q_{d1} + \Delta Q > Q_{do}$. There will be a backwater curve of the M_2 type.

(ii) $\Delta Q < Q_{do} - Q_{d1}$. The upstream discharge is $Q_o = Q_{d1} + \Delta Q < Q_{do}$. Now upstream of the intake the backwater curve will be of the M_1-type.

4.2.2. Application of fixed weir

In order to obtain a sufficiently high water level at the intake sometimes a fixed weir is installed. This is common practice on Java to withdraw irrigation water from a river. In this case the water level at the intake is discontinuous.

The bed level upstream of the weir is obviously influenced by the presence of the weir. In case all sediment passes eventually the weir ($t \rightarrow \infty$) the bed level can be estimated. This is the case if the sediment passes over the top of the weir or when it is flushed through the weir by special gates.

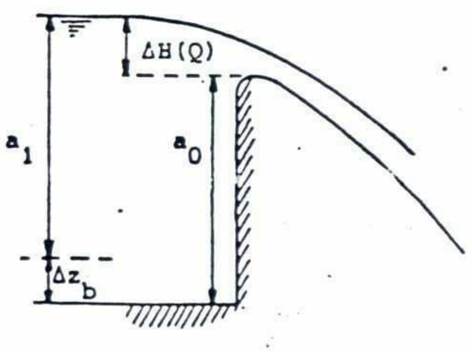


Fig. 4.4. Sedimentation upstream of fixed weir.

The yearly sediment transport in the old equilibrium situation amount to

$$V = \int_0^{\infty} S(Q) \cdot f_o(Q) \cdot dQ \qquad (4\text{-}7)$$

When this yearly amount passes the weir then the eventual average bed level rise (Δz_b) upstream of the weir can be found by expressing the transport formula as a function of the depth.

If a_o is the height of the crest of the weir above the original bed level then the depth (a_1) upstream of the weir follows from (Fig. 4.4):

$$a_1 = a_o + \Delta H - \Delta z_b \qquad (4\text{-}8)$$

in which the depth of flow (Δ H) over the weir is a (known) functions of the discharge (Q). Hence with $S = B\, m\, u^n$

$$V = \int_0^\infty B\, m\, \{\frac{Q}{B(a_o+\Delta H-\Delta z_b)}\}^n . f_o\{Q\}.dQ \qquad (4\text{-}9)$$

Combination of Eqs. (4-7) and (4-9) shows that Δz_b is the only unknown. Solution is possible e.g. by means of the *regula falsi*. Note that in this case f{Q} is the same in the old and the new situation. Naturally any suitable (real) sediment transport formula can be used.

4.2.3. Example: Morphological predictions Tana River

In Jansen (1979, p.433-440) a practical example is given regarding the morphological consequences of a proposed weir in the Tana River. The water is withdrawn for irrigation purposes.

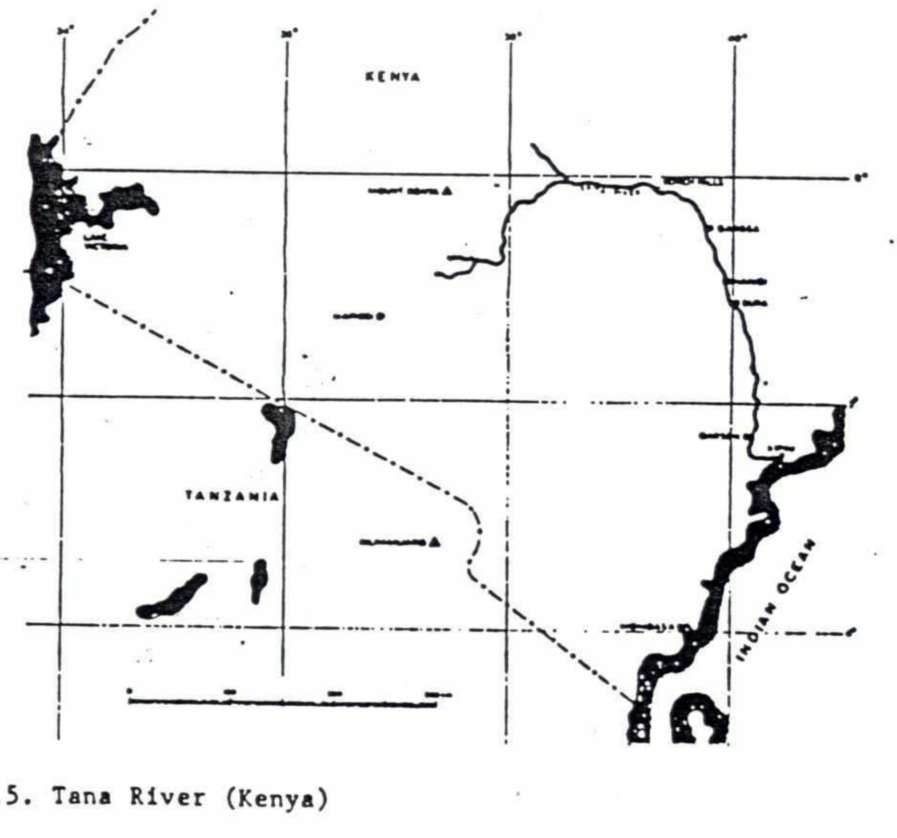


Fig. 4.5. Tana River (Kenya)

The computations were made with the Engelund-Hansen (1967) transport formula ($n = 5$) based on the availabe data ($D_{50} = 0.32$ mm; $i_o = 0.35 \cdot 10^{-4}$). Using Eq. (4-9) the eventual rise of the bedlevel upstream of the weir was estimated at $\Delta z_b = 2$ m for $a_o = 4$m.

Note: In Jansen (1979, p.437) the term (1 - dh/ds) in Eqs. (6.4-12), (6.4-13) and (6.4-15) has to be deleted.

As can be seen from Table 3.2 the Tana River is relatively fast. The main reason to carry out time-depending computations was the wish to get informed about $z_b(t)$ downstream of the weir. Qualitatively temporary erosion can there be expected. This is due to the fact that for small values of t a small sediment supply via the weir will be present due to the sedimentation *upstream* of the weir. The computations showed a temporary degradation of 3 to 4 m downstream of the weir to be reached after 6 months. Note that on top of this degradation *local scour* downstream of the weir may be present.

4.3. Withdrawal of sediment

In Sub-Section 3.2.4 the principle of the withdrawal of sediment from a river has been discussed with respect to the new equilibrium situation ($t \rightarrow \infty$) due to the continuous dredging of part of the sediment transported.

Now the problem will be treated in a more general sense. The problem can occur due to 'sand mining'. However, a similar problem can be present due to the subsidence of the river bed due to mining of gas, oil, coal, etc. in deep layers below the river bed. From the basic equations (see Sub-Section 3.2.1) the water equations are still valid. This holds also for the equations for the sediment transport.

The equation of continuity for the solid phase requires a change. This equation now reeds

$$\frac{\partial z_b}{\partial t} + \frac{\partial s}{\partial x} = W(x,t) \qquad (4\text{-}10)$$

The right-hand side represents a *source-term* describing the lowering of the bed due to subsidence. This term will be non-zero in a certain reach of a river.

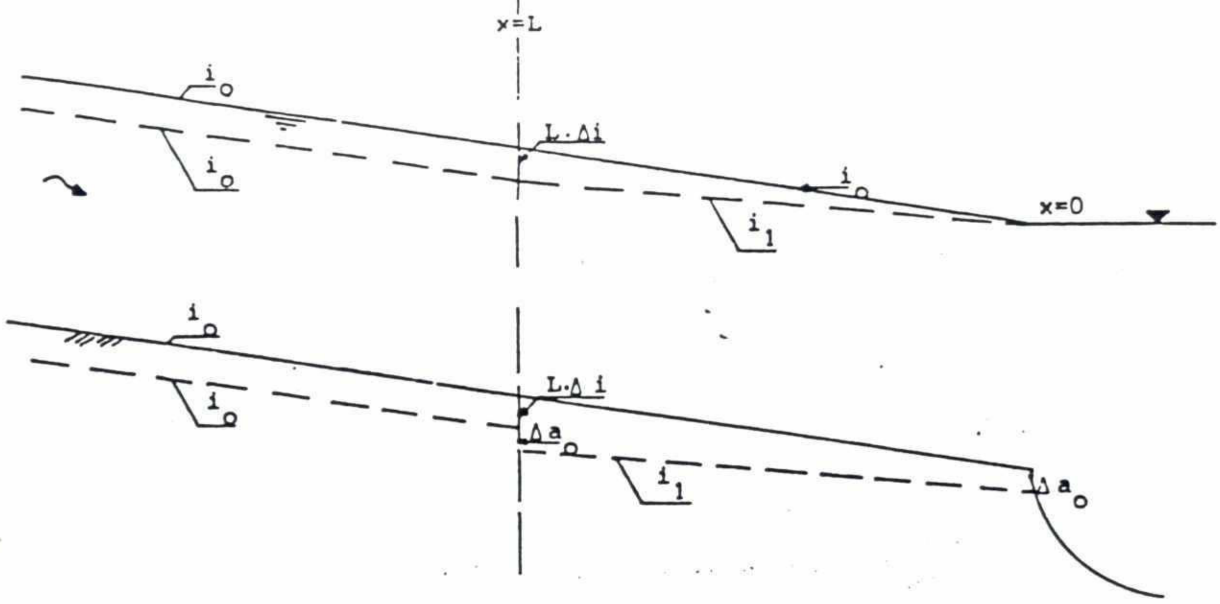


Fig. 4.6. Sediment withdrawal ($t = 0$ and $t \to \infty$)

In Fig. 4.6 the situation is sketched for continuous dredging of ΔS at $x = L$. As has been shown in Sub-Section 3.2.1 downstream of $x = L$ the slope will for $t \to \infty$ become flatter and the depth smaller. For a constant discharge the water level at $x = L$ will lower over a distance $\Delta h = L \cdot \Delta i = L(i_o - i_1)$.

In the interval $0 < x < L$ the depth will increase by Δa_o following from Eq. (3-49). Just downstream of $x = L$ the bed will finally have lowered over a distance $\Delta z_b(L,\infty) = \Delta h + \Delta a_o$. At $x = L$ the bed level is discontinuous. The bottom step is Δa_o. For all values $x > L$ both the bed levels and the water levels are lowered by Δh.

In Fig. 4.6 a mild positive bedslope has been assumed. Therefore the flow is subcritical. Note that the water level and bed level have to be drawn starting from the erosion-base (sea or lake level). This is the only point that remains the same for $t = 0$ and $t \to \infty$.

Remark:

(i) The situation of Fig. 4.6 is a rather theoretical one. In practice both Q and ΔS will be time-depending. However, to understand the results of time-depending morphological computations such a simplified case is of great help.

(ii) The lowering of water- and bed-levels due to withdrawal of sediment may have negative side-effects to other users of the river (e.g. for a water intake).

4.4. Constriction of width

Figure 4.6 gives also the general solution for $t \to \infty$ if the width is constricted in the interval $0 < x < L$. For the smaller width the slope becomes flatter and the depth larger. (See also Fig. 3.9).

To understand the morphological phenomena it is of importance to study the situation at $t = 0$ thus before any change of the bed has taken place.

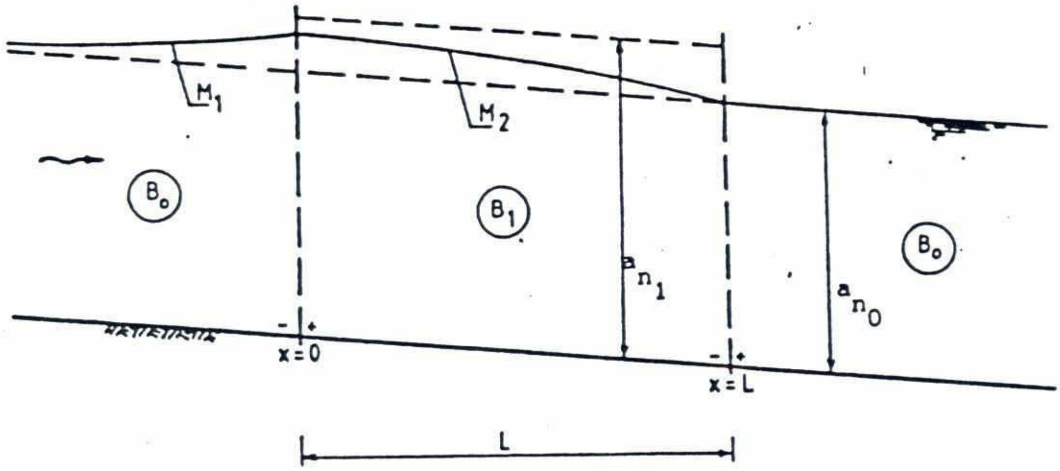


Fig. 4.7. Constriction of width: situation at t = 0.

In Fig. 4.7 the longitudinal profile is given. In the reach $0 < x < L$ the width has been reduced from B_o to B_1. Therefore the *normal depth* is there larger. For a mild positive slope backwater curve M_1 and M_2 are present as indicated in Fig. 4.7.

This gives regions of erosion and sedimentation. In Fig. 4.8 the function $S(x,o)$ is indicated qualitatively.

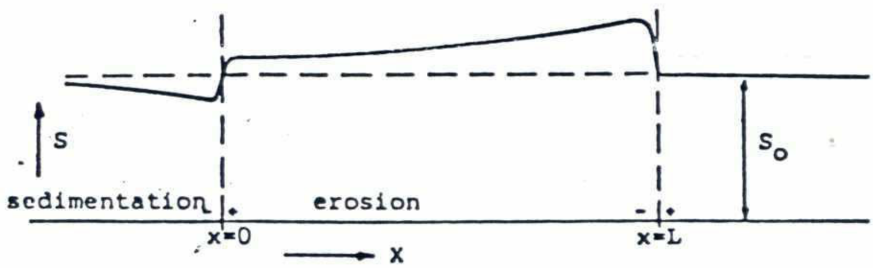


Fig. 4.8. Function S(x,o) due to constriction.

At $x = 0$ and $x = L$ there will be discontinuities in the bed level. To study these discontinuities it is of importance to realize that both the transport of water and sediment is continuous at $x = 0$ and $x = L$. However, the continuity of the sediment *cannot* be studied by

$$B \frac{\partial z_b}{\partial t} + \frac{\partial S}{\partial x} = 0 \qquad (4\text{-}11)$$

because $\partial z_b / \partial t$ is not defined at the discontinuities. However, the continuity equation holds in the *integral form* namely $S_- = S_+$. Moreover ofcourse $Q_- = Q_+$. Hence with $s = m u^n$ and $q = u.a.$

$$\frac{a_+}{a_-} = \left[\frac{B_-}{B_+} \right]^{\frac{n-1}{n}} \qquad (4\text{-}12)$$

As the water level is continuous the step in the bed level is equal to the difference $\Delta a = a_+ - a_-$

- For $x = 0$ the result is $a_-(o,t) < a_+(o,t)$. Hence there is a *downward* bottom step in the flow direction.
- For $x = L$ the result is $a_-(L,t) > a_+(L,t)$. This gives in the flow direction a step *upward* in the bottom.

The function $S(x,o)$ as indicated in Fig. 4.8 will lead to erosion and sedimentation. At $x > L$ temporary sedimentation takes place which has disappeared at $t \to \infty$. In the final situation, for a constant discharge Q_o the sediment transport follows from $S(x,\infty)$ = constant = S_o.

For $t \to \infty$ the following changes of the bed level compared to the old equilibrium are present

- $x > L$ — no change
- $0 < x < L$ — lowering $= (i_o - i_1)(L - x) + \Delta z'_b$
- $x < 0$ — lowering $= (i_o - i_1)L$

in which $\Delta z'_b$ follows from Eq. (3-52) and i_1 from Eq. (3-53).

4.5. Bend cutting

River navigation may be improved by cutting a sharp bend. In Fig. 4.9 a schematic example is given. A morphological computation has been carried out for a varying discharge, solving Eqs. (3-17) and (3-18).

In order to understand the morphological processes Fig. 4.10 indicates the principle. In this figure the water level differences are exaggerated. In the new bend (reach II) it is assumed that the bed level at t = 0 follows a straight line between the bed levels of the reaches I and III of the river. The length L_o of the original bend is shortened to L_1. Thus $i_{b_1} > i_{bo}$. As $q = Ca_n^{3/2}\, i_b^{1/2}$ it follows $a_{n_1} < a_{no}$. Hence at t = 0 backwater curves are present (Figs. 4.9 and 4.10).

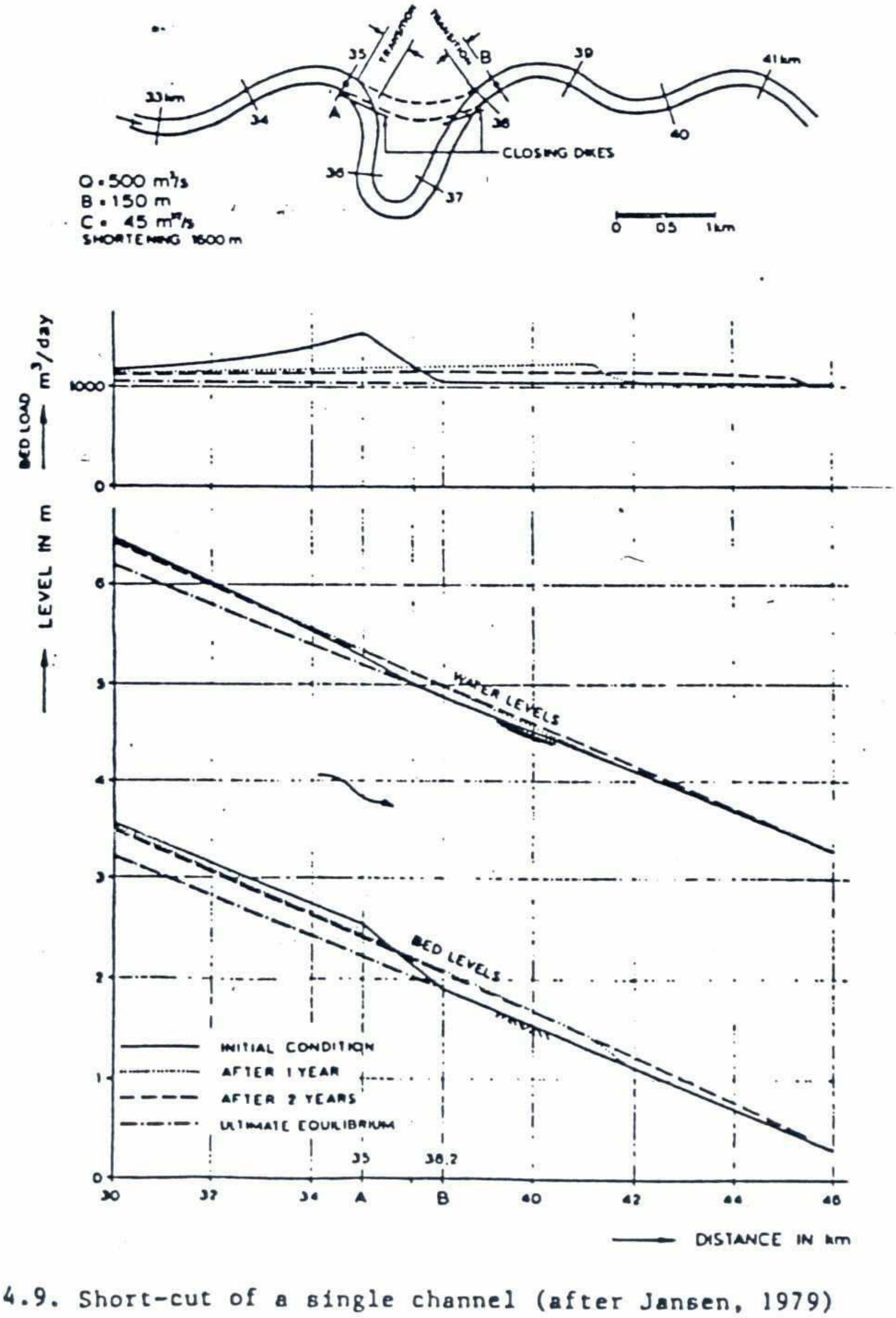


Fig. 4.9. Short-cut of a single channel (after Jansen, 1979)

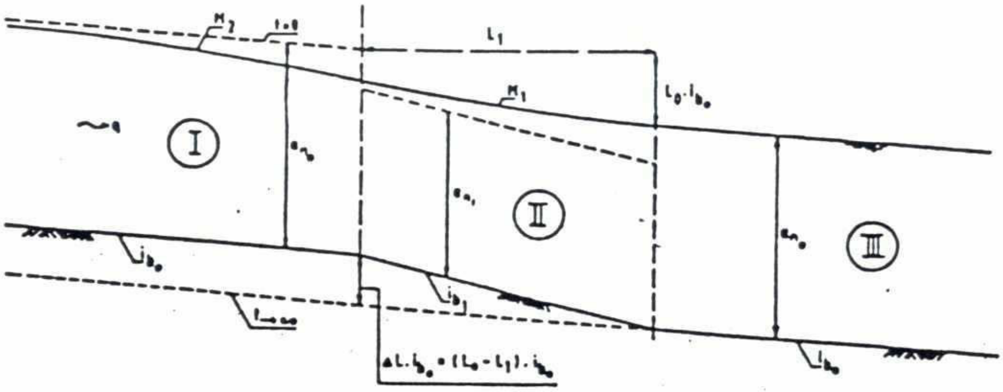


Fig. 4.10. Principle of bend-cutting (t = 0)

The situation for t = 0 cannot be an equilibrium one. The velocity gradients $(\partial u/\partial x)$ lead to transport gradients $(\partial s/\partial x)$. Hence $\partial z_b/\partial t \neq 0$.

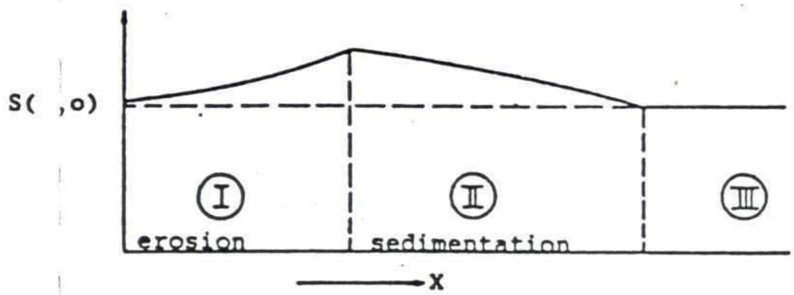


Fig. 4.11 Function S(x,o) for Fig. 4.10.

In Fig. 4.11 schematically the function S(x,0) is sketched for the case B(x) = constant. As $\partial z_b/\partial t + \partial s/\partial x = 0$ the reach I has erosion for t = 0. Sedimentation occurs in reach II (t = 0). For t > 0 temporary sedimentation occurs in reach III.

For reach II at $t \to \infty$ the original bed level is again present. For $t \to \infty$ the bed level in reach I has been lowered by $\Delta z_b = (L_o - L)i_b$.

Remark:

(i) For slow rivers dredging in the new bend will prevent the temporary reduction of the depth in reach III due to sedimentation. For quick rivers this is not necessary. One may even dredge along the alignment of the new bend a *pilot channel*. The river will then reach relatively soon the new situation.

(ii) Naturally the old bend will be closed by means of a dam preferably during low discharges. For Fig. 4.9 the computations have been carried out assuming that at t = 0 the dam was constructed.

4.6. Channel closure

In Fig. 4.12 an example is given, taken from Jansen (1979, p.348) regarding the increase of depth for navigation by the closure of one river branch.

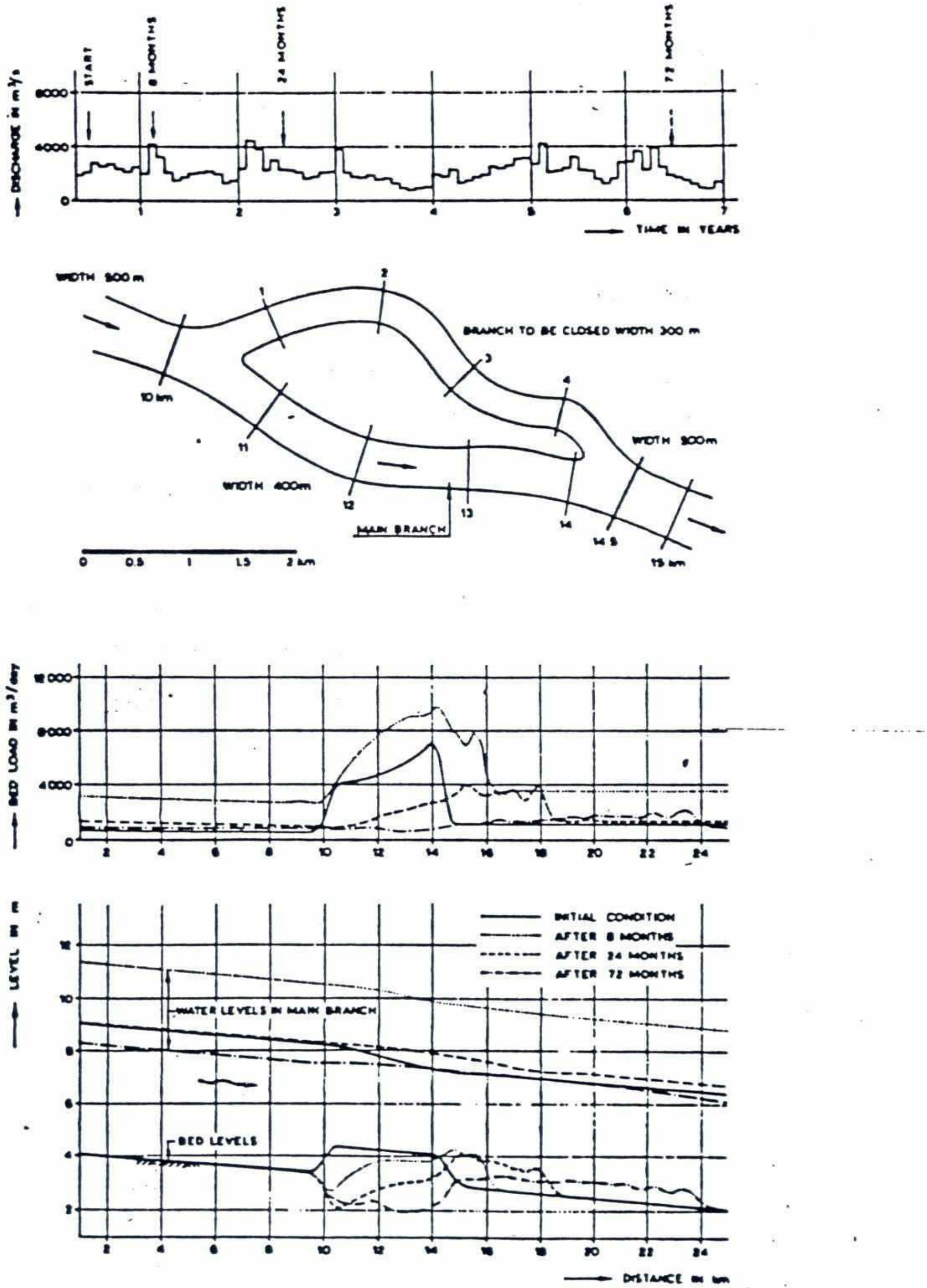


Fig. 4.12. Closure of a branch (after Jansen, 1979)

In this case the river is flowing around an island. The narrow branch is closed in order to arrive *eventually* at the situation that the other branch is deepended. In the example of Fig. 4.12 the discharge varies in time.

To understand the behaviour of $z_b(x,t)$ first the situation for a constant discharge is considered.
For $t < 0$ at $x = 0$ (the bifurcation) and at $x = L$ (the confluence) a step in the bed level Δz_o will be present. This step cannot be computed as it depends on the distribution of the sediment at the bifurcation (see Section 2.4). Also the slopes in the two branches for $t < 0$ depend in the distribution of water and sediment at the bifurcation.

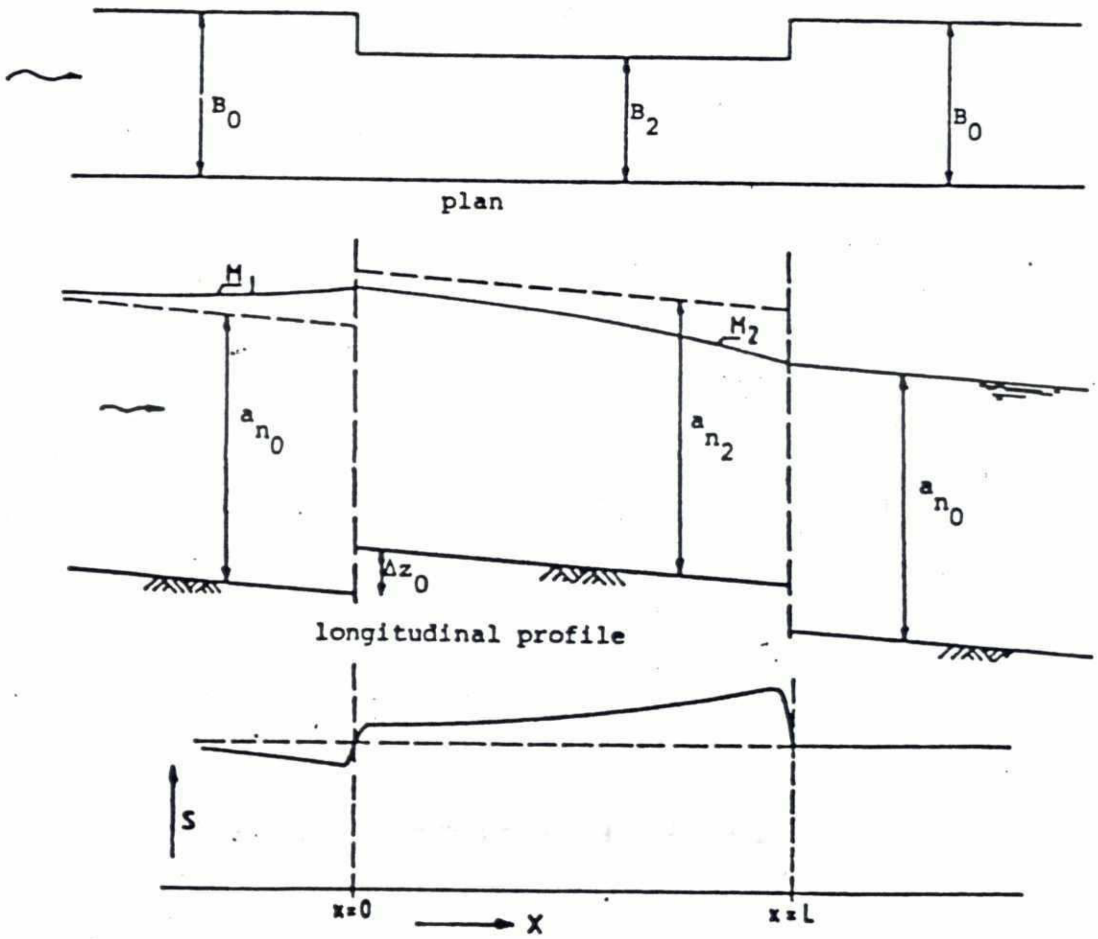


Fig. 4.13. Principle of channel closure ($t = 0$).

In Fig. 4.13 the situation at $t = 0$ is given; the minor branch has just been closed. The essential difference with Fig. 4.10 is that here the bottom steps Δz_o are present; they originate from the situation at $t < 0$.

Besides these two steps there are two more steps present due to the change in width at $x = 0$ and $x = L$ for $t > 0$. These are the steps treated in Sub-Section 3.2.4 (Case III).

Thus for $t > 0$ there are *four* bottom steps present:

- For $x = 0$: (i) The step because $B_o > B_1$. This step remains at $x = 0$ for $t > 0$.
  (ii) The step Δz_o originates from $t < 0$. This propagates downstream for $t > 0$. It is an *expansion wave*, so it becomes flatter for $t > 0$.
- For $x = L$: (iii) A step because $B_2 < B_o$, this step stays at $x = L$.
  (iv) A step Δz_o from the situation at $t < 0$. This discontinuity propagates downstream for $t > 0$. It is a shockwave.

To understand the behaviour of step (ii) and (iv) reference can be made to Fig. 3.4 where the deformation of a hump is sketched.

Inspection of Fig. 4.13 shows that $z_b(x,t)$ indeed contains the four bottom steps indicated above.

Remarks:

(i) As was explained in Section 4.4 the presence of discontinuities in the width at $x = 0$ and $x = L$ makes that the *differential* equation expressing the sediment continuity does not apply at the discontinuities because $\partial z_b/\partial t$ is not defined there. The continuity of sediment is expressed in *integral* form because $S(x,t)$ is continuous.

(ii) Figure 4.12 shows that the water depth downstream of the island is temporary decreased due to sedimentation. Apparently the example regards a relatively slow river. In practice it may be advisable to 'help' the river by means of dredging. Otherwise the benefit of the local river improvement will be obtained only after some time.

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Main symbols

symbol	description	dimensions
a	depth	[L]
A	area	[L^2]
B	width	[L]
c	celerity	[LT^{-1}]
C	Chézy-coefficient	[$L^{\frac{1}{2}}T^{-1}$]
D	grain diameters	[L]
Fr	Froude number	[-]
g	acceleration of gravity	[LT^{-2}]
h	water level	[L]
H	energy head	[L]
i	(energy) slope	[-]
k_N	Nikuradse sandroughness	[L]
L	length	[L]
M	mass	[M]
p	pressure	[$ML^{-1}T^{-2}$]
q	discharge per unit width	[L^2T^{-1}]
Q	discharge	[L^3T^{-1}]
r	radius of curvature	[L]
R	hydraulic radius	[L]
s	sediment transport per unit width (bulk volume)	[L^2T^{-1}]
S	sediment transport (bulk volume)	[L^3T^{-1}]
t	time	[T]
u	flow velocity in x-direction	[LT^{-1}]
v	flow velocity in y-direction	[LT^{-1}]
w	flow velocity in z-direction	[LT^{-1}]
W	fall velocity	[LT^{-1}]
x	horizontal coordinate	[L]
X	transport parameter = $s/ D^{3/2}\sqrt{g\Delta}$	[-]
y	horizontal coordinate	[L]
Y	flow parameter = $\Delta D / \mu a i$	[-]
z	vertical coordinate	[L]

$z_{(b)}$	bed level	[L]
Z	= W/ku_*	[-]
Δ	relative density sediment = $(\rho_s - \rho)/\rho$	[-]
ε	turbulent viscosity	$[L^2T^{-1}]$
η	= z/a = relative depth	[-]
κ	von Kármán constant	[-]
Λ	= xi/a = length scale river	[-]
μ	ripple factor	[-]
ν	kinematic viscosity	$[L^2T^{-1}]$
ρ	density of water	$[ML^{-3}]$
ρ_s	density of sediment	$[ML^{-3}]$
τ	shearstress	$[ML^{-1}T^{-2}]$
ϕ	sediment concentration	[-]

'Engineering Potamology' 1985

Corrections

page 3 line 8 from bottom: has *read* had

page 6 last line: demped *read* damped

page 7 last sentence, *read:* In Table 1.1 the rivers are listed by the size of the catchment area.

page 13 line 8 from bottom: finding *read* findings

page 18 line 12 from top: This a *read* This is a

page 29 first line, formula *read:* $Q = BCa\sqrt{ai_b}$

page 31 third line from bottom: transport unit *read* transport per unit

page 56 line 7 from bottom: *read* (1979, p. 59).

page 61 line 12 from top: parameter *read* parameters

page 70 line 11 from top: (Q) *read* (ΔQ)

line 12 from top: (x L) *read* ($x < L$)

line 5 from bottom: *read* $i_{b_1} > i_{b_0}$

line 3 from bottom: *read* (a_{01}/a_{00}).

$$Q = BC h^{3/2} i^{1/2} \quad ; \quad B = Q C^{-1} h^{-3/2} i^{-1/2}$$

$$S = B D^{-p} m C^{n} h^{n/2} i^{n/2}$$

$$S D^{p} = Q C^{-1} h^{-3/2} i^{-1/2} m C^{n} h^{n/2} i^{n/2}$$

$$= m Q C^{n-1} h^{\frac{n-3}{2}} \cdot i^{\frac{n-1}{2}} \; :$$

$S D^{p} :: Q h^{\frac{n-3}{2}} i^{\frac{n-1}{2}}$ → eliminate B (1)

$S D^{p} :: B^{\frac{3-n}{3}} Q^{n/3} i^{n/3}$ → eliminate h (2)

$S D^{p} :: B^{1-n} Q^{n} h^{-n}$ → eliminate i (3)

S, D, Q = const.

$h_1 < h_0$

$h^{\frac{n-3}{2}} i^{\frac{n-1}{2}}$ = const. from (1)

$h^{n-3} i^{n-1}$ = const.

$$\left(\frac{i_1}{i_0}\right)^{n-1} = \left(\frac{h_0}{h_1}\right)^{n-3}$$

$$\frac{i_1}{i_0} = \left(\frac{h_0}{h_1}\right)^{\frac{n-3}{n-1}} \; \ldots$$

$B^{1-n} h^{-n}$ = const.

$$\left(\frac{B_1}{B_0}\right)^{1-n} = \left(\frac{h_0}{h_1}\right)^{n}$$

$$\frac{B_1}{B_0} = \left(\frac{h_0}{h_1}\right)^{\frac{n}{1-n}}$$ ... from (3)

Do

Elementary River Mechanics

Improved Lane's balance

1 Derivations

Assume steady, uniform conditions; discharge, sediment and hydraulic roughness transport constant.

Basic equations:

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

$$s\;S = m\,u^{n} \qquad (2)$$

$$s\;S = D^{-p}\,m_1\,u^{n}$$

$$S = B\,D^{-p}\,m_1\,u^{n} = B\,D^{-p}\,m_1\,C^{n}\,(\,hi\,)^{\frac{n}{2}}$$

D = grain size

Solutions

(a) Solve by eliminating h :

$$Q\,i = B\,C\,h^{\frac{3}{2}}\,i^{\frac{3}{2}}$$

$$hi = \left[\frac{Qi}{BC}\right]^{\frac{2}{3}}$$

$$S\,D^{p} = B\,m_1\,C^{n}\left[\frac{Qi}{BC}\right] = B^{1-\frac{n}{3}}\,m_1\,C^{\frac{2n}{3}}\,Q^{\frac{n}{3}}\,i^{\frac{n}{3}}$$

$$S\,D^{p} :: B^{\frac{3-n}{3}}\,Q^{\frac{n}{3}}\,i^{\frac{n}{3}} \qquad (3)$$

(b) Solve by eliminating i :

$$Q = B\,C\,h\,(\,hi\,)^{\frac{1}{2}}$$

$$hi = \left[\frac{Q}{B\,C\,h}\right]^{2}$$

$$S = B\,D^{-p}\,m_1\,C^n\,(hi)^{\frac{n}{2}} = B\,D^{-p}\,m_1\,C^n\left[\frac{Q}{B\,C\,h}\right]^n$$

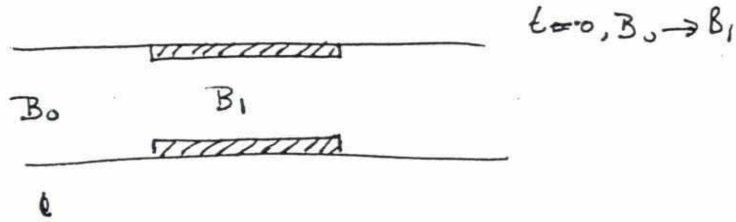
$$S\,D^p = B^{1-n}\,m_1\,Q^n\,h^{-n}$$

$$S\,D^p :: B^{1-n}\,Q^n\,h^{-n} \qquad (4)$$

2 Applications

(1) Narrowing

$B_o \rightarrow B_1$, hence
Q, S and D constant :



(a) Change in slope:

$$i^{\frac{n}{3}}\,B^{1-\frac{n}{3}} = constant$$

$$\frac{i_1}{i_o} = \left[\frac{B_1}{B_o}\right]^{\frac{n-3}{n}} \qquad (5)$$

(b) Change in depth:

$$h^{-n}\,B^{1-n} = constant$$

$$\frac{h_1}{h_o} = \left[\frac{B_o}{B_1}\right]^{\frac{n-1}{n}} \qquad (6)$$

(2) Withdrawal of water

$Q_o \rightarrow Q_o - \Delta Q$, hence

S, B and D constant

(a) Change in slope:

$$Q\,i = constant$$

$$\frac{i_1}{i_o} = \frac{Q_o}{Q_o - \Delta Q} \qquad (7)$$

(b) Change in depth:

$$Q^n h^{-n} = constant$$

$$\frac{h_1}{h_o} = \frac{Q_o - \Delta Q}{Q_o} \qquad (8)$$

(3) Mining of sediment

$S_o \rightarrow S_o - \Delta S$, hence

Q, B and D constant

(a) Change in slope:

$$S :: i^{\frac{n}{3}}$$

$$\text{or} \quad S\, i^{-\frac{n}{3}} = constant$$

$$\frac{i_1}{i_o} = \left(\frac{S_o - \Delta S}{S_o} \right)^{\frac{3}{n}} \qquad (9)$$

(b) Change in depth:

$$S :: h^{-n}$$

$$\text{or} \quad S\, h^n = constant$$

$$\frac{h_1}{h_o} = \left(\frac{S_o}{S_o - \Delta S} \right)^{\frac{1}{n}} \qquad (10)$$

(4) Water withdrawal including change of width

Including effect on width via regime equation for width, which reads as:

$$B :: Q^{0.5}$$

(a) Change in slope:

$$S\, D^p :: (Q^{0.5})^{\frac{3-n}{3}} Q^{\frac{n}{3}} i^{\frac{n}{3}}$$

or

$$S\, D^p :: Q^{\frac{n+3}{6}} i^{\frac{n}{3}}$$

n8

for water with drawal $Q_o \rightarrow Q_o - \Delta Q$, hence S and D are constant

$$Q^{\frac{n+3}{6}}\, i^{\frac{n}{3}} = constant$$

$$\frac{i_1}{i_o} = \left[\frac{Q_o}{Q_o - \Delta Q}\right]^{\frac{n+3}{2n}} \qquad (11)$$

$$\text{For } n = 5 \rightarrow \frac{i_1}{i_o} = \left[\frac{Q_o}{Q_o - \Delta Q}\right]^{\frac{4}{3}}$$

$$n = 3 \rightarrow \frac{i_1}{i_o} = \frac{Q_o}{Q_o - \Delta Q}$$

(b) Change in depth:

$$S\, D^p :: (Q^{0.5})^{1-n}\, Q^n\, h^{-n}$$

$$S\, D^p :: Q^{\frac{1+n}{2}}\, h^{-n}$$

for water withdrawal $Q_o \rightarrow Q_o - \Delta Q$, hence S and D are constant

$$Q^{\frac{1+n}{2}}\, h^{-n} = constant$$

$$\frac{h_1}{h_o} = \left[\frac{Q_o - \Delta Q}{Q_o}\right]^{\frac{1+n}{2n}} \qquad (12)$$

$$\text{For } n = 5 \rightarrow \frac{h_1}{h_o} = \left[\frac{Q_o - \Delta Q}{Q_o}\right]^{\frac{3}{5}}$$

$$n = 3 \rightarrow \frac{h_1}{h_o} = \left[\frac{Q_o - \Delta Q}{Q_o}\right]^{\frac{2}{3}}$$

Hence same tendency as for constant width, but less serious effect

Measure	Influence on slope	Influence on depth
Q, B, C and D constant		
General	$S\,D^p :: B^{\frac{3-n}{3}}\,Q^{\frac{n}{3}}\,i^{\frac{n}{3}}$	$S\,D^p :: B^{1-n}\,Q^n\,h^{-n}$
Narrowing $B_0 \rightarrow B_1$	$\frac{i_1}{i_o} = \left(\frac{B_1}{B_o}\right)^{\frac{n-3}{n}}$	$\frac{h_1}{h_o} = \left(\frac{B_o}{B_1}\right)^{\frac{n-1}{n}}$
Water withdrawal ΔQ	$\frac{i_1}{i_o} = \frac{Q_o}{Q_o - \Delta Q}$	$\frac{h_1}{h_o} = \frac{Q_o - \Delta Q}{Q_o}$
Sediment mining ΔS	$\frac{i_1}{i_o} = \left(\frac{S_o - \Delta S}{S_o}\right)^{\frac{3}{n}}$	$\frac{h_1}{h_o} = \left(\frac{S_o}{S_o - \Delta S}\right)^{\frac{1}{n}}$
Q variable ($p\{Q\}$), B, C and D constant $\{p\,\{Q\}$ = probability distribution of Q$\}$		
Water withdrawal ΔQ	$\frac{i_1}{i_o} = \left[\frac{\int_0^\infty Q^{\frac{n}{3}}\,p_o\,\{Q\}\,dQ}{\int_0^\infty Q^{\frac{n}{3}}\,p_1\,\{Q\}\,dQ}\right]^{\frac{3}{n}}$	$\frac{h_1}{h_o} = \left[\frac{\int_0^\infty Q^n\,p_1\,\{Q\}\,dQ}{\int_0^\infty Q^n\,p_o\,\{Q\}\,dQ}\right]^{\frac{1}{n}}$
Q, C and C constant, B according to regime equation Q :: $B^{0.5}$		
General	$S\,D^p :: Q^{\frac{n+3}{6}}\,i^{\frac{n}{3}}$	$S\,D^p :: Q^{\frac{1+n}{2}}\,h^{-n}$
Water withdrawal ΔQ	$\frac{i_1}{i_o} = \left(\frac{Q_o}{Q_o - \Delta Q}\right)^{\frac{n+3}{2n}}$	$\frac{h_1}{h_o} = \left(\frac{Q_o - \Delta Q}{Q_o}\right)^{\frac{1+n}{2n}}$

Table Overview of response of river system to river engineering measures

ND

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

DRAFT FINAL REPORT
PLANNING STUDY

VOLUME II

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

DECEMBER 1992

7 RESPONSE OF A RIVER SYSTEM TO AFPM

7.1 GENERAL

One of the important aspects when considering the application of AFPM for the Jamuna River is the estimation of the response of the river system. This response can be divided into:

(i) response at short notice, and

(ii) the long term consequences for the river characteristics.

In this respect also a differentiation in space can be made. Responses at short notice usually are limited in distance at which their effect is noticeable, while long term consequences will in due time become apparent over the whole distance of the river.

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

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

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

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

7.2 IMPOSED AND DEPENDENT VARIABLES

7.2.1 General

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

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

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

7.2.2 Variables and Equations

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

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

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

(1) continuity equation Q = Bhu (undisputed)
(2) momentum equation which in its simplest form (steady uniform conditions) takes the form of the Chézy equation or the Manning equation (undisputed),
(3) roughness predictor (only approximately known),
(4) sediment transport predictor (only approximately known),
(5) definition of sinuosity $p = i_v/i$ (by definition, hence undisputed),
(6) relation between the hydraulic radius and the width, depth and some other parameters (only approximately known).

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

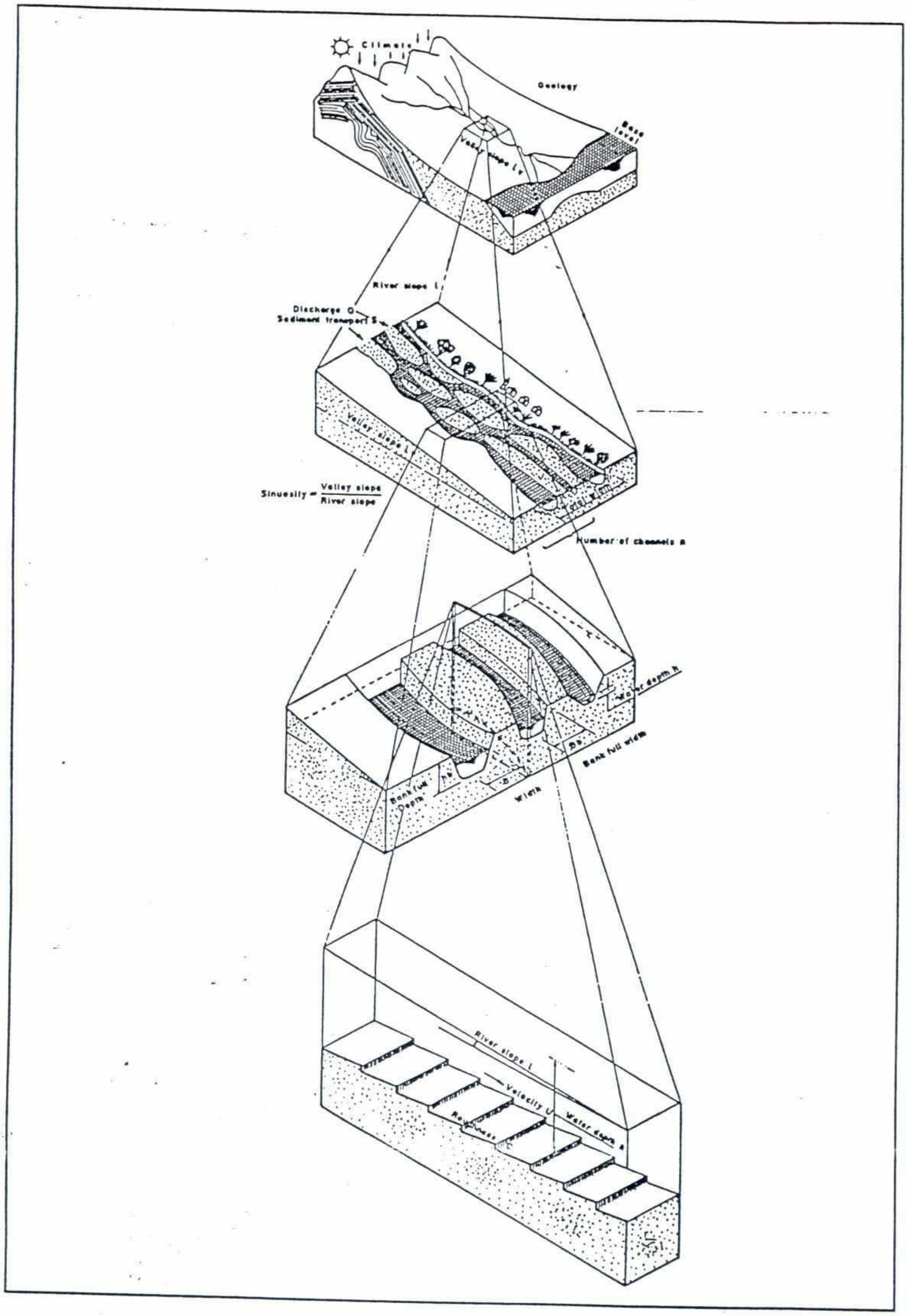


Fig. 7.2-1: Main parameters in a river system

7.2.3 Dependent Variables and AFPM

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

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

- Option 1 — Close only very aggressive channels, probably some 2 or 3 per year.
- Option 2 — Try to reduce the total width of the braided belt by consequently closing the most outward channels, and by forcing artificial cutoffs.
- Option 3 — Construct gradually more bank protection works at vulnerable places, which will lead to a narrower channel (conform what has been the case downstream of Sirajganj over the last decades).
- Option 4 — Try to create a transition of the braided river with multiple channels to a meandering river with one channel.
- Option 5 — As option 4 but in combination with river training works to stabilize the alignment of the river.

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

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

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

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

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

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

7.3 PREDICTION METHODS FOR NATURAL RIVERS

7.3.1 General

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

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

7.3.2 Width Predictors

7.3.2.1 General

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

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

7.3.2.2 Empirical Predictors

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

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

$$P = 2.67\, Q^{\frac{1}{2}} \quad (7.3\text{-}1)$$

$$R_h = 0.473\, Q^{\frac{1}{3}} f^{-\frac{1}{3}} \quad (7.3\text{-}2)$$

$$i = \frac{1}{1750} f^{\frac{5}{3}} Q^{-\frac{1}{6}} \quad (7.3\text{-}3)$$

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

$$f = 1.76 \sqrt{D_{50}} \quad (7.3\text{-}4)$$

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:

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

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

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

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

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

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

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

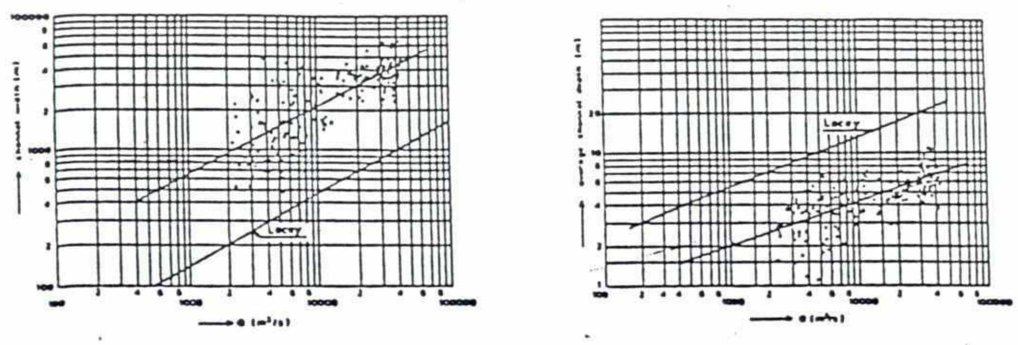
Relations for braided sand bed rivers are given by Klaassen & Vermeer (1988). In fact these equations were derived from an analysis of cross-sections from the Jamuna River, and some spurious correlation was introduced, because the bankfull discharge was divided over the channels according to their conveyance.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

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

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

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

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



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

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

(Source: Klaassen & Vermeer, 1988)

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

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

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

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

7.3.2.3 Theoretical Width Predictors

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

(1) Stable channel approach
(2) Lateral exchange approach
(3) Extremal hypotheses.

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

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

This approach assumes that the channel carries little or no sediment transport. It may therefore be assumed that for all particles, both in the bed and in the banks, the critical conditions are not exceeded. The critical shear stress is of course a function of the particle

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size (according to Shields (1936) and of the cohesion). Furthermore the effect of the slope of the banks on the stability has to be accounted for. Finally also the lateral variation of the shearstress in a channel has to be taken into account. The maximum shear stress on the banks for a trapezoidal cross-section with fairly steep slopes, is usually only about 75 % of the shear stress on the bed. Taking these factors into account it is possible to derive theoretical cross-sectional shapes that satisfy stable conditions. Related approaches have been followed by Thorne (1982), relating the channel stability to the stability of the banks, and Singh (1983), relating the shear stress (reduced according to the reasoning above) on the banks to the limiting maximum shear stress. As shown by Bettess et al. (1987) these approaches lead to results that are not compatible with the empirical relations discussed before. Furthermore these approaches cannot be used for rivers.

Re (2) Lateral exchange approach

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

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

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

Re (3) Extremal hypotheses

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

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

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

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

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

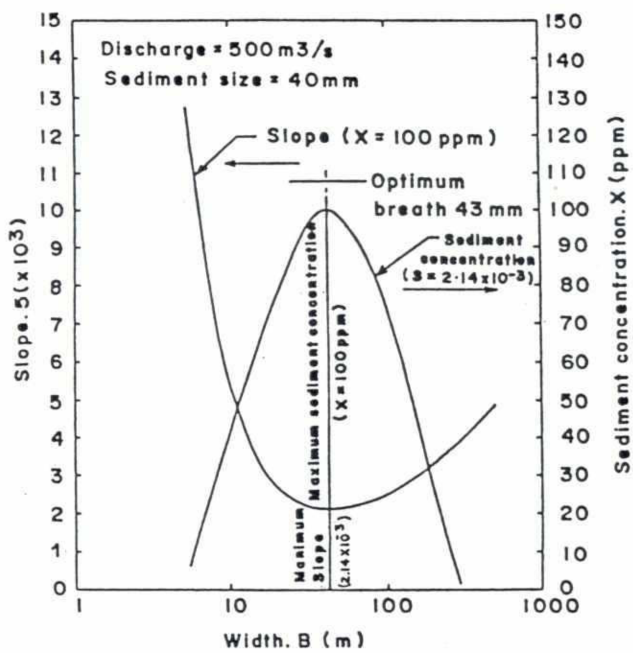

Fig. 7.3-2: **Example of variation of slope and sediment concentration as a function of width**

(Source: White et al, 1982)

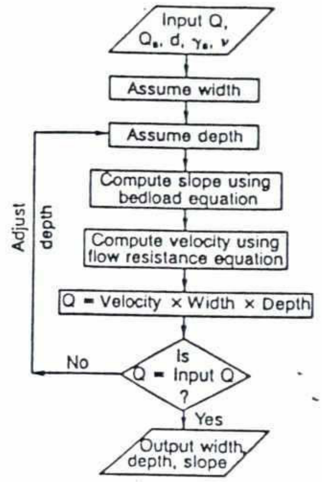

Fig. 7.3-3: Flow chart showing major steps in computation of Chang (1979)

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

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

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

$$z = 0.5 \qquad \textit{if} \quad Q < 1\ m^3/s \qquad (7.3\text{-}10)$$

$$z = 0.5\ Q^{\frac{1}{4}} \qquad \textit{if} \quad Q > 1\ m^3/s \qquad (7.3\text{-}11)$$

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

Table 7.3-1: Comparison of different extremal hypotheses proposed

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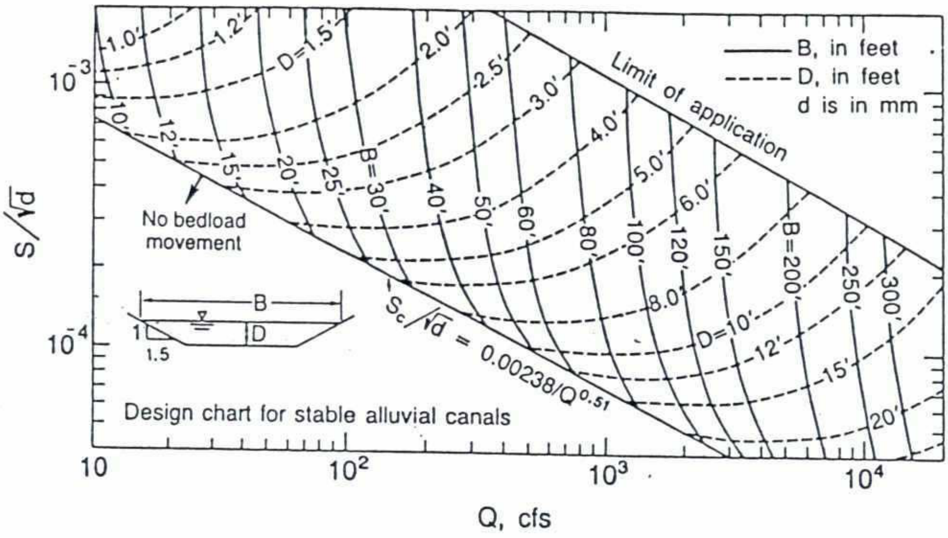
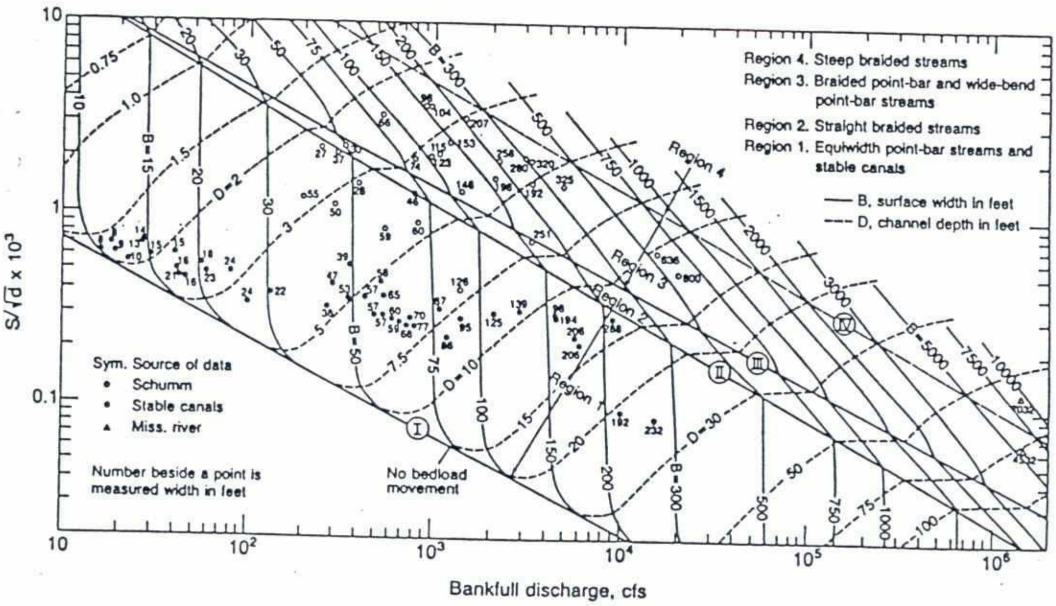


(a) Stable channels



(b) Sand-bed channels

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

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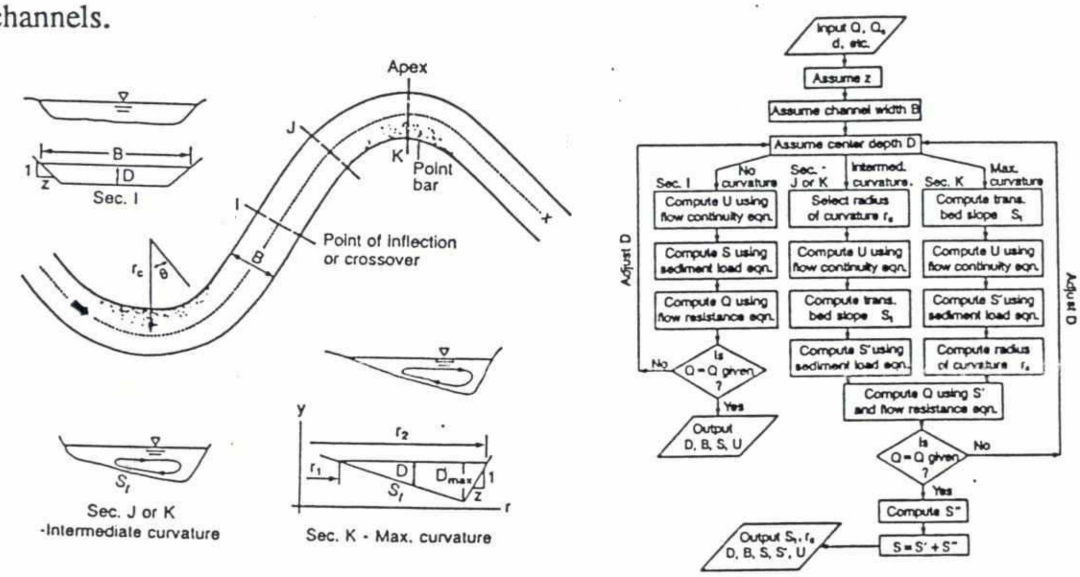
Different researchers have elaborated the use of the extremal hypothesis up to different levels:

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

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

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

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



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

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

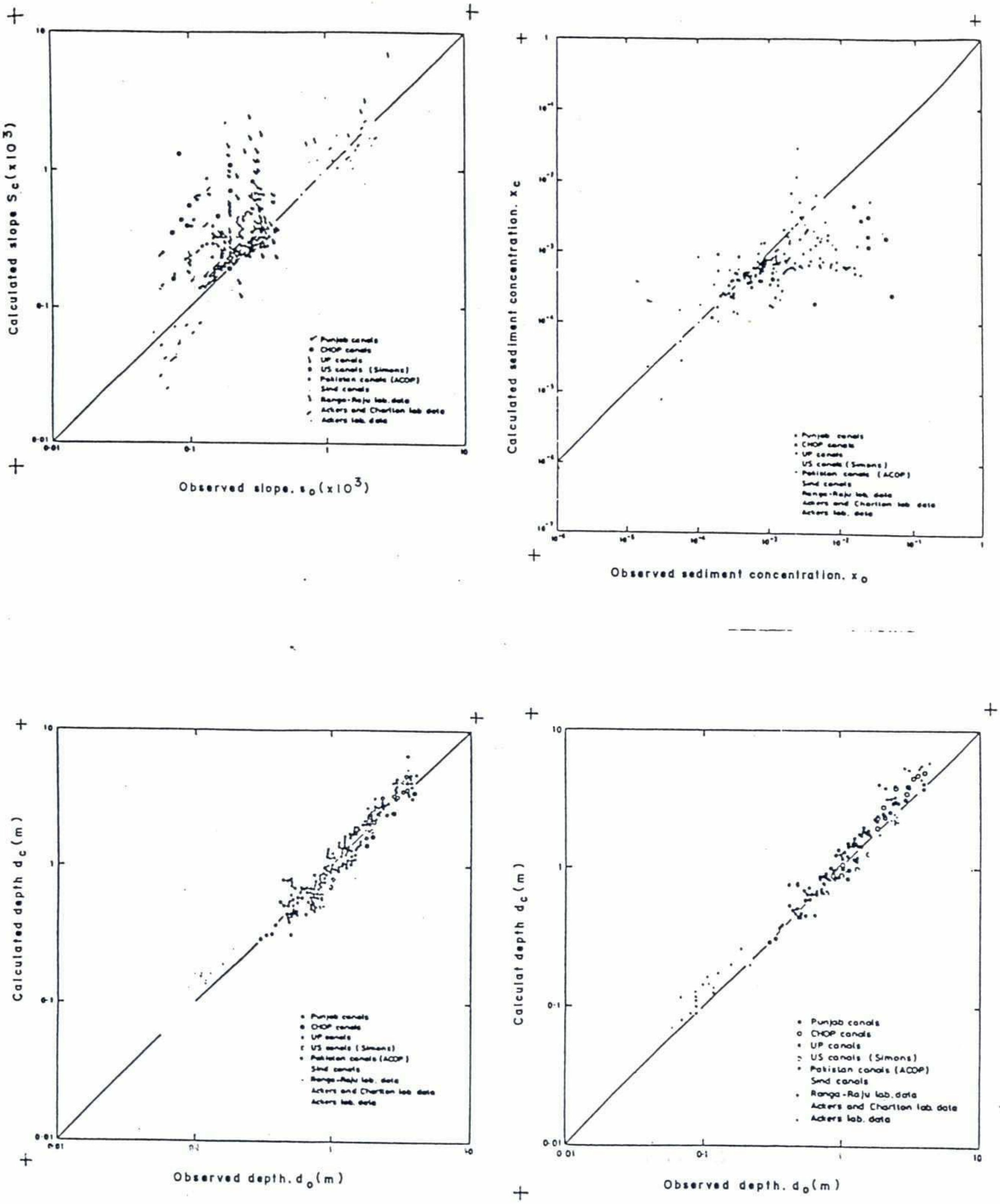


Fig. 7.3-6: Comparison of maximum sediment transport method of White et al with (mostly) field data

(Source: White et al, 1982)

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

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

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

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

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

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

which are amazingly similar to the empirical relations.

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

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

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

7.3.3 Predictors for the Sinuosity

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

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

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

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

(1) $i = i_v$:

The slope of the river corresponds to the slope of the valley: the river will remain straight.

(2) $i > i_v$:

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

(3) $i < i_v$:

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

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

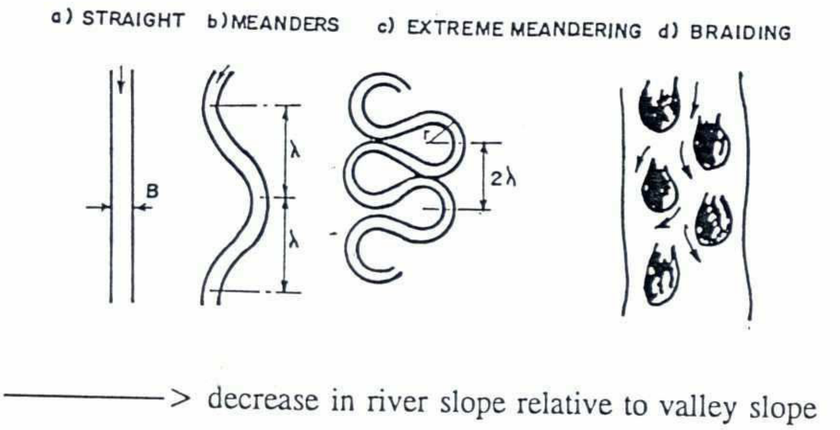

Fig. 7.3-7: Meandering as a way to cope with a difference between valley slope and river slope

(Source: Ramette, 1990)

7.3.4 Predictors for the Number of Channels

7.3.4.1 General

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

7.3.4.2 Transition from Meandering to Braiding

(a) Empirical methods

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

$$i = 0.0116\ Q^{-0.44} \qquad (7.3\text{-}17)$$

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.

$$\lambda_w = \frac{C^2}{2g} h_0 \qquad (7.3\text{-}18)$$

for water motion, and

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

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

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

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

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

7.3.4.3 Prediction of Number of Channels in a Braided System

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

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

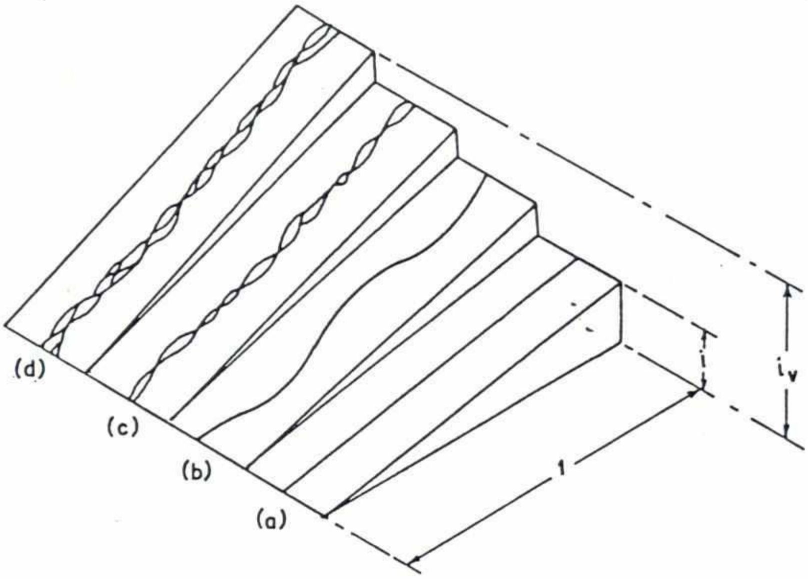

Fig. 7.3-8: Channel pattern for increasing valley slope
(Source: Bettess & White, 1983)

Hereafter prediction methods for the number of channels in a braided river system are described briefly. A difference is made between:

(a) empirical relations, and
(b) theoretical predictions.

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

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

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

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

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

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

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

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

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

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

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

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

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

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

7.3.5 Predictors for the Total Width of the River

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

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

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

7.4 SELECTION OF PREDICTION METHOD

7.4.1 Present Status of Prediction Methods

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

7.4.2 Testing

Considering the above an important step in the selection procedure is the testing of the applicability of the various proposed methods against the characteristics of the Jamuna river. This holds in particular for the prediction of (1) the width of the river system, (2) the sinuosity, (3) the number of braids and (4) the total width of the river system. Some empirical predictors are available, either especially developed for the Jamuna river or potentially applicable. From these methods a selection has to be made to identify the methods that are most suitable for use within the present Project. Considering the above, an important step in the selection procedure is the verification of the applicability of the various proposed methods against data on the Jamuna river. The characteristics of the Jamuna river have been determined on the basis of satellite images. Some results of this analysis are presented in the Fig. 7.4-1 (notably the total width of the river and the width of the channels as a function of the chainage) and Fig. 7.4-2 (providing a relation between the number of channels and the total width of the river).

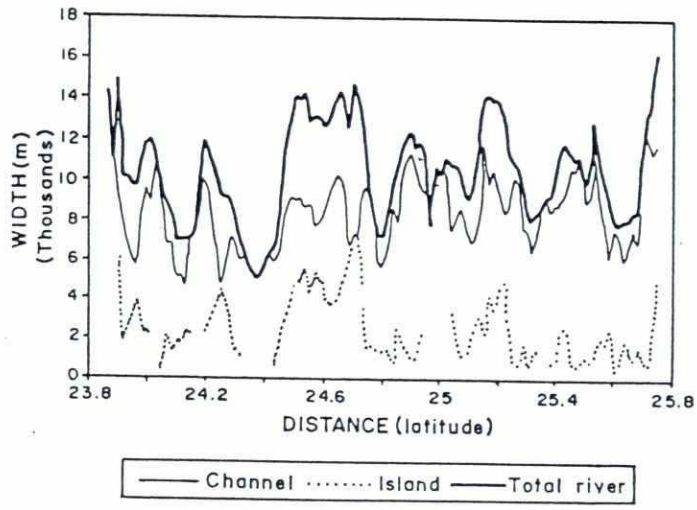

Fig. 7.4-1: Total width and combined width of all channels versus chainage of the Jamuna river for the year 1989

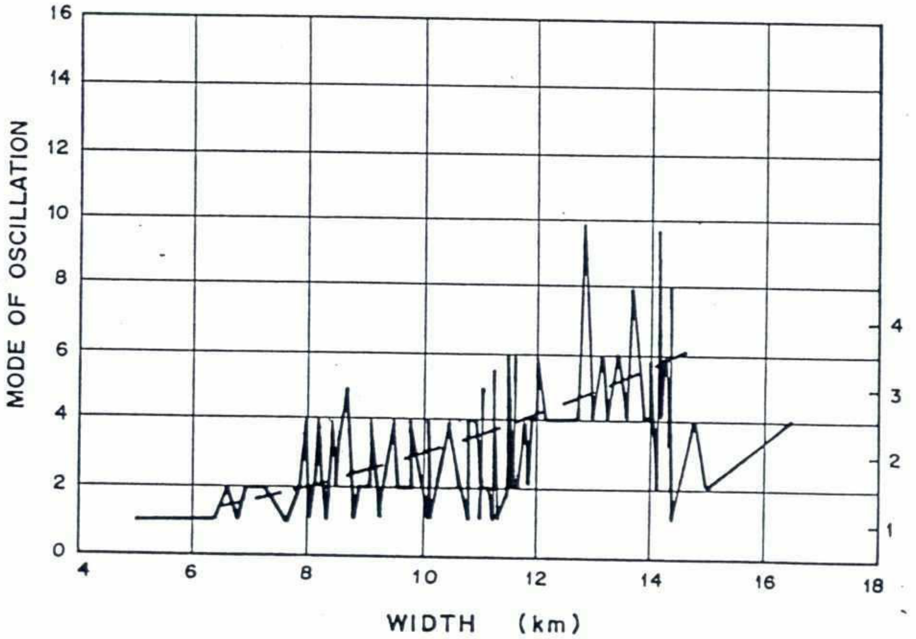

Fig. 7.4-2: Number of channels versus the total width of the Jamuna river

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

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

Re (1) Width of Channels

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

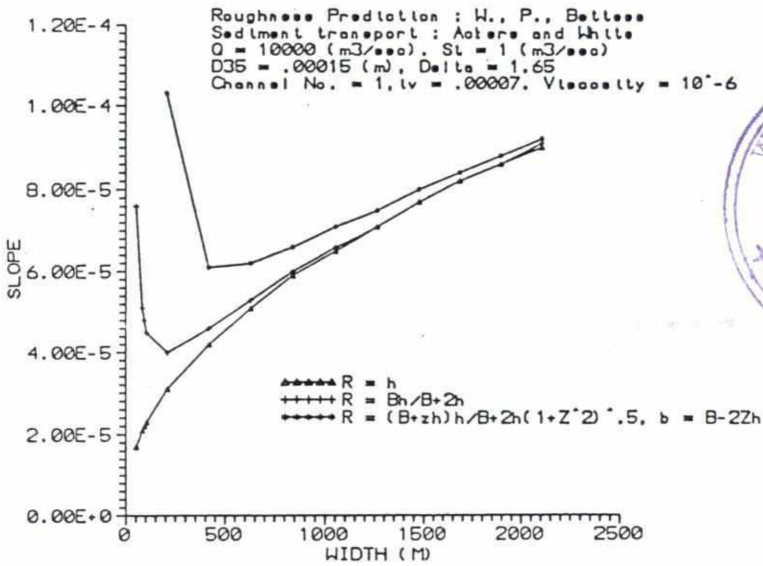

Fig. 7.4-3: **Testing of external hypothesis using the Ackers & White sediment transport predictor and the White et al. roughness predictor**

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

Re (2) Sinuosity

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

Re (3) Number of Channels

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

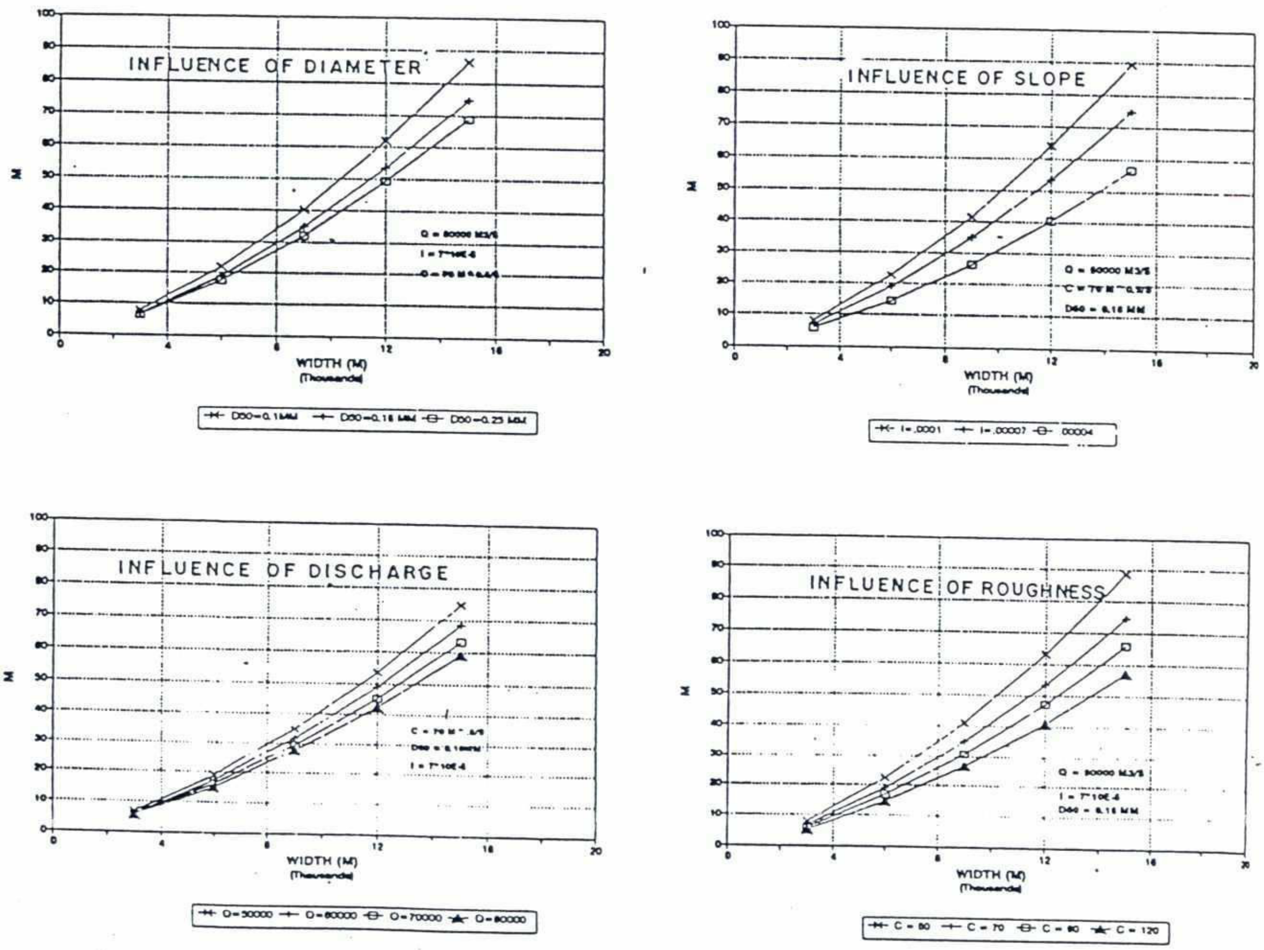

Fig. 7.4-4: Testing of prediction method for number of channels based on the marginal stability curve

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

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

Re (4) Total width

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

7.4.3 Development of Prediction Method for Braided Rivers

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

The method applied here can be summarized as follows:

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

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

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

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

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

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

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

(b) the sediment transport equation via:

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

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

and

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

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

$$U_b = C \sqrt{h_b \, i_b} \qquad (7.4\text{-}6)$$

(d) a hydraulic roughness predictor:

$$C = C \,(h, i_b, \Delta, D, \dots) \qquad (7.4\text{-}7)$$

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

$$B_b = 16.1 \left[\frac{Q_b}{k} \right]^{0.53} \qquad (7.4\text{-}8)$$

(2) Prediction of the sinuosity via:

$$p = \frac{i_v}{i_b} \qquad (7.4\text{-}9)$$

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

$$B_t = 2\, k\, A + B_b \qquad (7.4\text{-}10)$$

where

$$\lambda = 10\, B_b \qquad (7.4\text{-}11)$$

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

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

and

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

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

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

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

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

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Hence

$$B_b = f\,(\,Q_b\,) \qquad (7.4\text{-}14)$$

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

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

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

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

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

$$\lambda = 10.1\; B_b^{1.01} \qquad (7.4\text{-}15)$$

Here a simplified expression was used, reading

$$\lambda = 10\; B_b \qquad (7.4\text{-}16)$$

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

$$\lambda = f \left[\frac{Q_b}{k} \right]^{0.53} \qquad (7.4\text{-}17)$$

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

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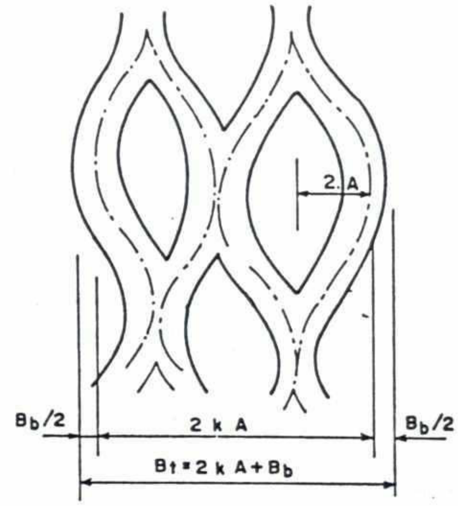

Fig. 7.4-5: Geometrical relation for the determination of the (minimum) total width

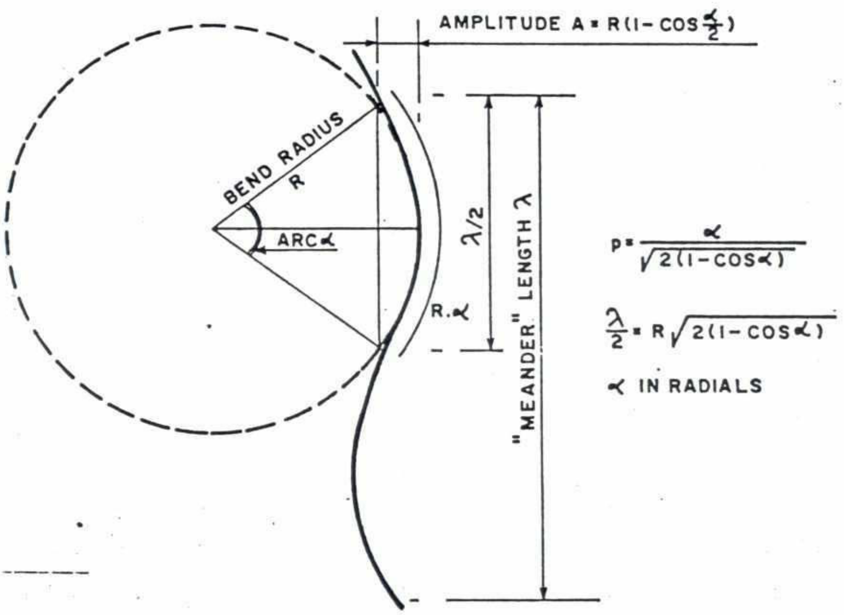

Fig. 7.4-6: Relation between amplitude and sinuosity for a curved channel

7.4.4 Calibration and Verification

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

- bankfull discharge Q_b;

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

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

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

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

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

is about 5 m for 3 parallel channels.

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

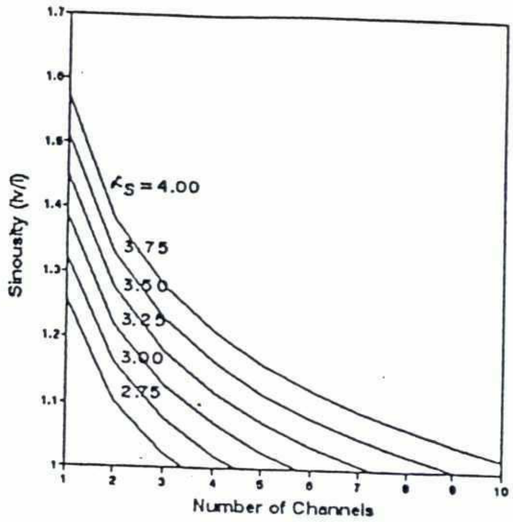
In a further step an attempt was made to verify the method proposed here. In first instance the conditions in the Jamuna river downstream of Sirajganj were simulated wit the proposed method, but in addition holding the value of λ_Q equal to 2.75. For the independent variables the following values were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

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

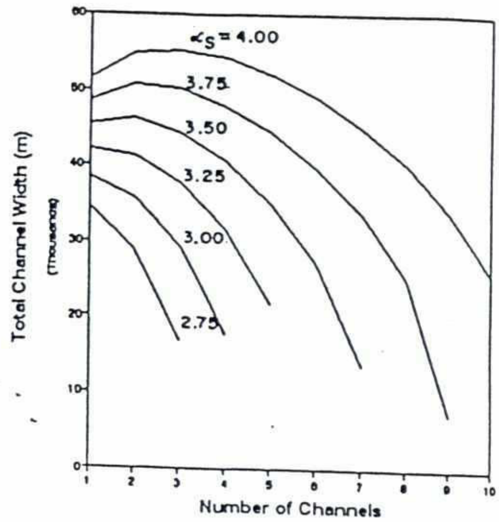
For these particular conditions it was predicted that the Jamuna river downstream of Sirajganj would have only 1 channel, with a meanderlength of some 90 km and a meander belt width of some 20 km. Comparing this with the actual conditions (2 channels on the average and a belt width of some 10 km, it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

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

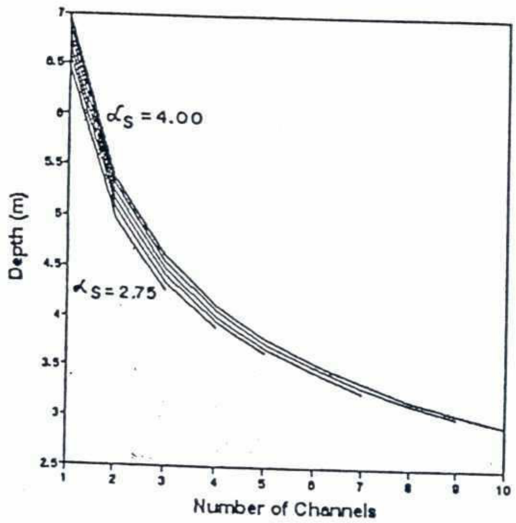
Finally an attempt was made to apply the prediction method to other rivers in Bangladesh. It was found, however, that the uncertainty as to the sediment load of these rivers, in combination with the possibility that different expressions for the different emperical equations should be used, did not allow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



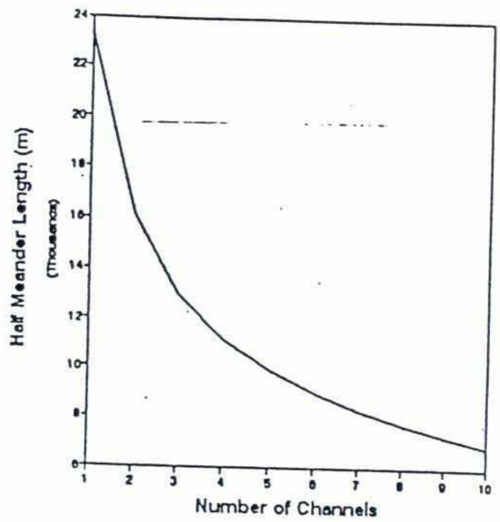
(a) Sinuosity



(b) Total channel width



(c) Average depth of individual channels



(d) Meander length

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

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

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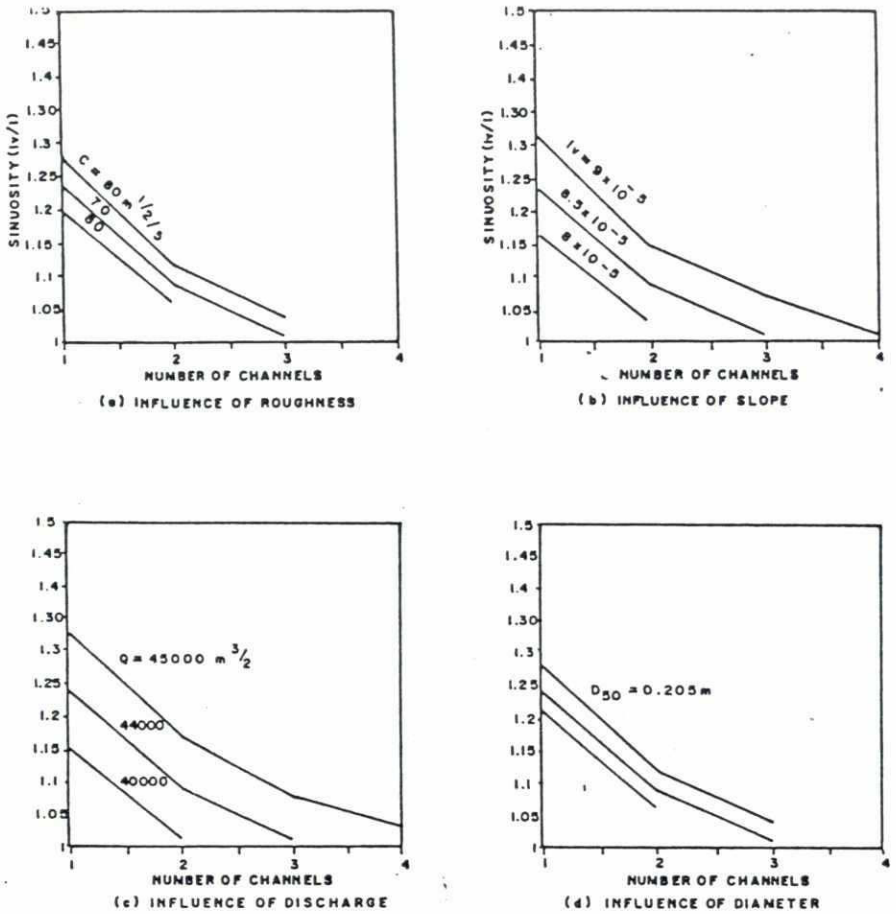

Fig. 7.4-8: Sensitivity of prediction for Bahadurabad for hydraulic roughness, valley slope, bankfull discharge and characteristic particle size

7.5 APPLICATION OF THE SELECTED METHOD

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

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

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

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

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

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

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

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

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

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

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

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

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

REFERENCES

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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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5 RIVER BENDS

5.1 Flow in bends

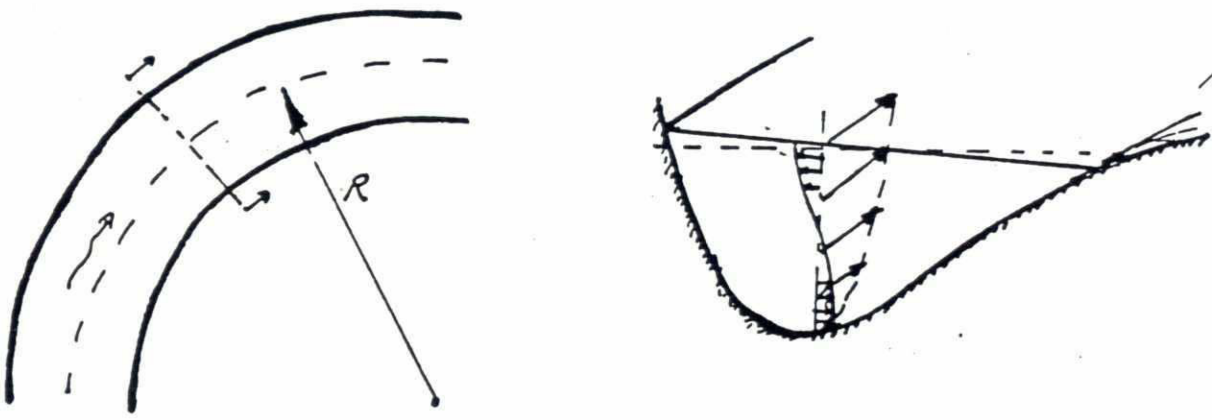

Fig. 5.1 Helicoidal flow

For axially symmetric flow in a bend, a lateral pressure gradient $\alpha \rho \overline{U}^2/R$, or a lateral slope $\partial h/\partial r$ equal to $\alpha . \overline{U}^2/gR$ will be generated. The value of α is slightly larger than 1 due to the non-uniformity of the velocity in the vertical direction. A good estimate is $\alpha = 1.06$ (Van Bendegom, 1947 and Jansen, 1979).

Water particles near the surface have a velocity larger than $\overline{U}$, the average velocity, and will therefore tend to follow a curve with R_s/R. Near the bed $U < \overline{U}$ and $R_b < R$, because the lateral pressure grandient is the same for all particles. This leads to the well-known spiral or helicoidal flow pattern in bends which has a very large influence on the bed profile as it will transport sediment from the outer bend towards the inner bend.

For axially symmetric flow and an assumed logarithmic velocity distribution over the depth, a simple analysis leads to an expression for the lateral velocity distribution (Rozovskii 1957, De Vriend 1977), almost linear over depth. The direction of the velocity near the bed, relative to the direction of the average velocity is given by

$$\tan \delta = - \frac{2}{\kappa^2} \cdot \frac{h}{R} \left(1 - \frac{\sqrt{g}}{C}\right)$$

where h = depth

κ = von Kármán constant

C = Chezy coefficient.

For a range of Chezy values from 30 to 60 $m^{½}/s$, tanδ varies from 9.5 h/R to 11 h/R with 10 h/R as a good average. This also means that the lateral component of the bed shear stress can be given as

$$\tau_y/\tau_x = 10\ h/R \qquad \text{where } \tau_x = \rho g h i$$

It must be stressed however that this analysis is only valid for fully developed spiral flow without effects of the banks. In reality there is an effect of friction along the banks and of the interaction between secondary flow and main flow, giving a phase lag between spiral flow and geometry (De Vriend, 1981).

5.2 Bend profiles

The spiral flow in bends will tend to transport particles towards the inner bend until a lateral slope is formed and an equilibrium between lateral components of bed shear stress forces and gravity forces is obtained (Van Bendegom, 1947)

$$\tau_y \cdot \frac{\pi}{4} D^2 = \sin\beta \frac{\pi}{6} D^3 (\rho_s - \rho) g$$

Taking $\tau_y = 10(h/r)\tau_x$ where $\tau_x = \rho g h i = \rho U_*^2$, this leads to

$$\sin\beta = 15 \frac{U_*^2}{\Delta g D} \frac{h}{R} \qquad \text{(a)}$$

Odgaard (1984) reviews theories by Kennedy, Falcon and himself and finds for uniform bed material

$$\sin\beta = 4.8 \sqrt{\theta} \frac{\bar{U}}{\sqrt{\Delta g D}} \frac{h}{R} \qquad \text{(b)}$$

in which θ is the Shields parameter for beginning of motion.

Experimental data are not sufficiently accurate to give a clear choice for

(a) or (b) but indicate some preferene for (b). Both relations show that the lateral slope is a function of flow velocity and depth and can therefore change during floods.
If it is assumed that the slope is proportional to r^{-1} and that (a) is valid (Jansen 1979) it can be shown that

$$\frac{dh}{h^2} = 15 \frac{R_o i}{\Delta D} \frac{dr}{r^2}$$

or

$$\frac{1}{h} - \frac{1}{h_o} = \left(\frac{1}{r} - \frac{1}{R_o}\right) \frac{15 R_o i}{\Delta D}$$

from which, for a given h_o and R_o other depths in the bends can be calculated. Apmann (1972) gave a simplified analysis, assuming the velccity to be constant. This leads to

$$h = a.r^n$$

For one river, the Buffelo Creek, he found $n = 2.5 \pm 0.6$.
These relations are not of much practical use however, because uniform bed material is assumed, whereas it known that the material in the outer bend is generally much coarser than in the inner bend due to the greater effect of the lateral shear force on the smaller grains.

Fig. 5.2 (Odgaard, 1981) shows the large variation in grain sizes and flow velocity in a practical case.

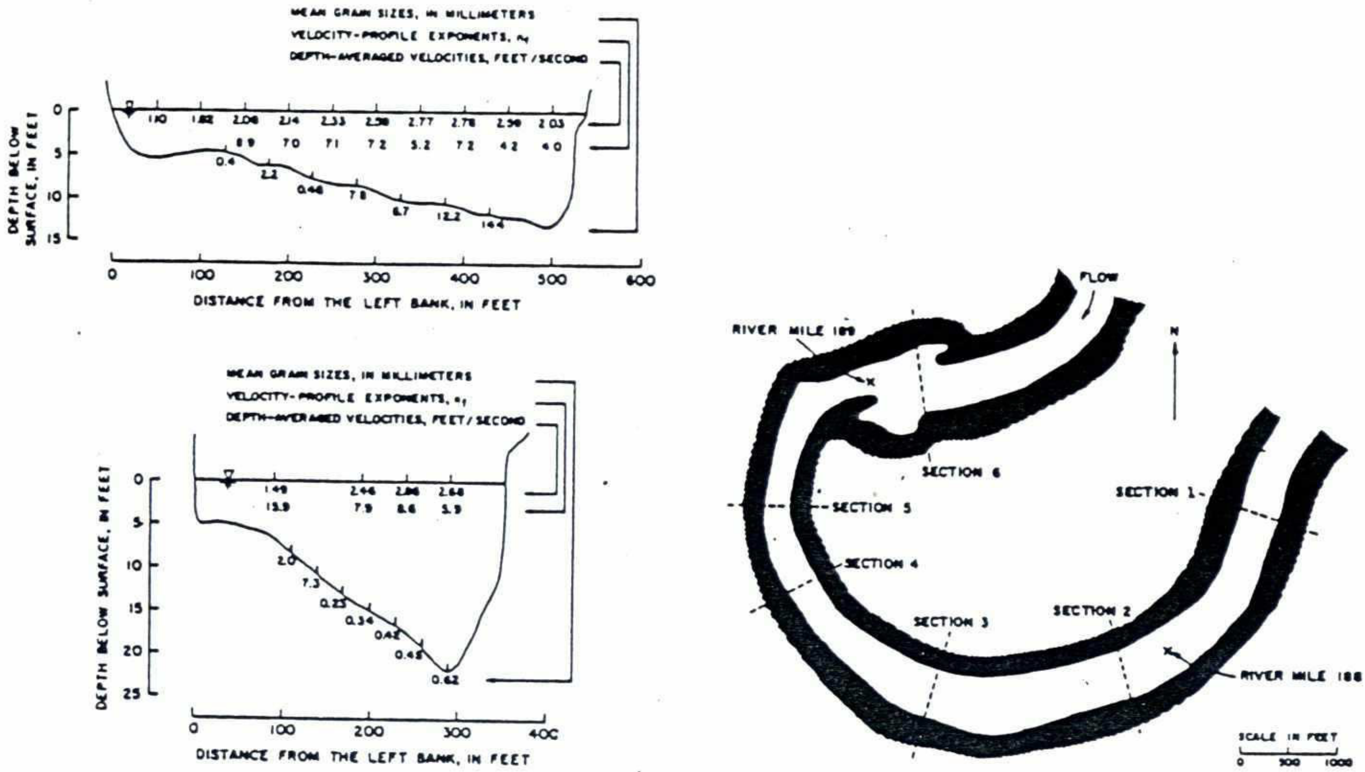

Fig. 5.2 Sacramento River bend.

Developments are now in the direction of a two-dimensional modelling of the flow and morphology of a river bend. Using the flow model developed by De Vriend (1977, 1981), Struiksma et al. (1985) compute the bed topography in bends with fixed side walls. Results agree quite well with experiments in a laboratory flume.

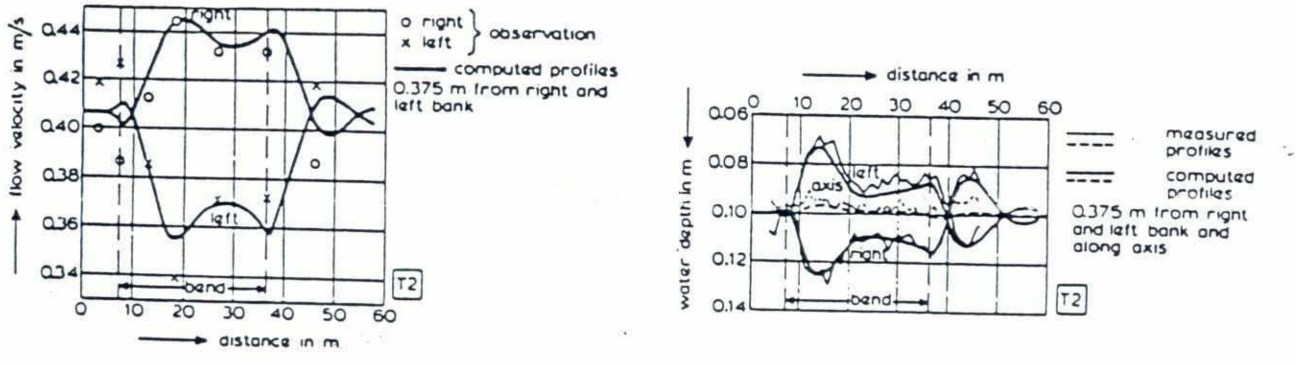

Fig. 5.3 Comparison of velocities and bed levels in a bend

5.3 Bank erosion

Bank erosion is a common feature of alluvial rivers. Rates of lateral erosion for various rivers are greatly different due to variations in geological structure and sedimentological composition of the valley material (see Fig. 5.4).

Rates of lateral migration vary from a few meters per year (Brice 1984, USA rivers) to 50-75 m/year (Missouri, Mississippi) and even 2500 m/year (Kosi River, India, see Garde and Ranga Raju, 1977). Coleman (1969) reports values of 30 to 750 m/year for the Brahmaputra River, occuring mainly during the flood season.

Bank erosion is most prominent in riverbends due to the increase of velocities in the outer bend and the spiral flow which tends to deepen the outer bend. Bank erosion can be due to scouring, undermining and subsequent soil-mechanical failure or liquefaction by overpressure in fine sand during falling water levels. Coleman (1969) observed that for the Brahmaputra River the majority of the failures was due to current undermining and subsequent failure of the levee deposits.

(a) A stream begins to downcut, forming terraces in former alluvial fill.
(b) Several episodes of valley cutting and fill. Original bedrock valley was filled with alluvium (1) has undergone two other cycles of erosion and sedimentation (2) and (3). Fill is currently being removed by the river.
After Leopold and Miller (1954)

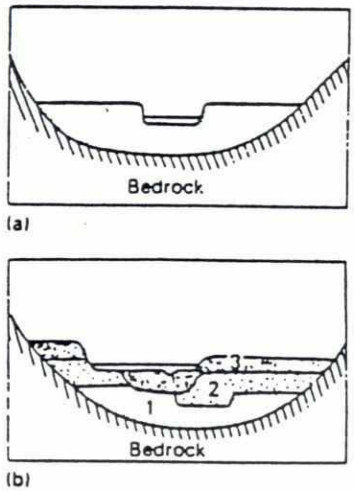

Fig.5.4 River valley formation.

There is not much theoretical or even empirical work related to quantitative prediction of bank erosion rates. Brice (1984) found that bank erosion rate increased linearly with drainage area for a number of USA rivers.

Hickin and Nanson (1984) did an extensive study using photographs of sand and gravel rivers in Western Canada. They found that erosion rate was a function of the radius of curvature to width ratio R/W, with a maximum at R/W = 2.5. An empirical relation (see Fig. 5.5) was:

$$1 < R/W < 2.5 \qquad f(R/W) = 2/3\ (R/W-1)$$

$$R/W > 2.5 \qquad f(R/W) = 2.5\ W/R$$

The maximum erosion rate occurs for R/W = 2.5 and is defined as $M_{2.5}$ (in m/year) and increases with total streampower Ω:

$$\Omega = Q_5\ \tau\ h^{-1} = \rho g\ Q_5\ I \qquad \text{(Watt/m')}$$

where Q_5 = discharge exceeded once in 5 years.

$M_{2.5}$ is inversely proportional to a bank-strength parameter G_B (dimension N/m^2) which is a function of bed material size (see Fig. 5.6).

$$M(R/W) = M_{2.5}\ .\ f(R/W)$$

$$M_{2.5} = \frac{\Omega}{h\ G_B}$$

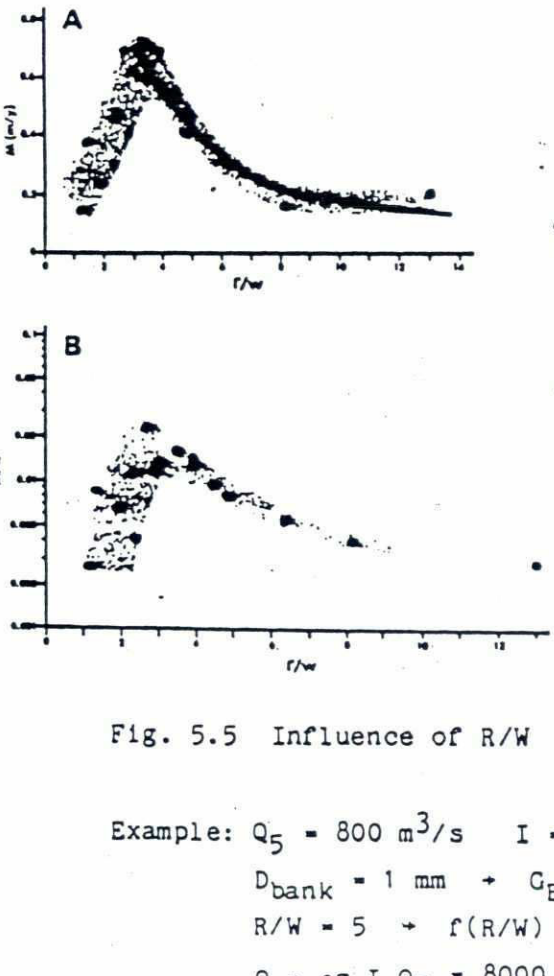

Fig. 5.5 Influence of R/W

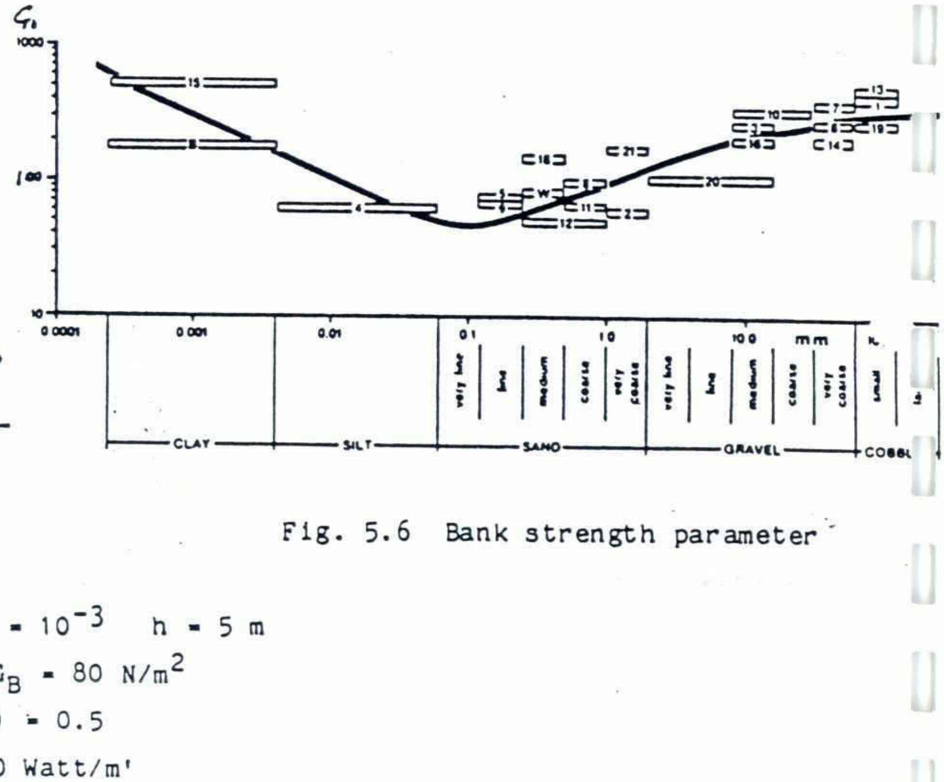

Fig. 5.6 Bank strength parameter

Example: Q_5 = 800 m^3/s $\quad I = 10^{-3} \quad h = 5$ m

D_{bank} = 1 mm $\rightarrow$ G_B = 80 N/m^2

R/W = 5 $\rightarrow$ f(R/W) = 0.5

$\Omega = \rho g\ I\ Q_5$ = 8000 Watt/m'

$M_{2.5} = \Omega .\ h^{-1} G_B^{-1}$ = 20 m/year

$M = M_{2.5}\ .\ f(R/W)$ = 10 m/year.

The use of this relation in other areas must be done with care but can give some idea of possible bank erosion rates.

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6 MORPHOLOGICAL COMPUTATIONS

6.1 Introduction

To analyse the changes in rivers due to man-made or natural influences, mathematical models can be used under certain conditions. For example, if the bed is fully alluvial (no non-erodible parts) and the banks are fixed or only slowly changing, one can use a one-dimensional schematisation to compute bed and water level changes.
For river bends and bifurcations, two-dimensional models are being developed but also here the banks have to be fixed.

6.2 One-dimensional models

Assuming a constant width of the river, the following relations for the water motion, sediment transport and bed level changes are used (de Vries 1973, Jansen 1979).

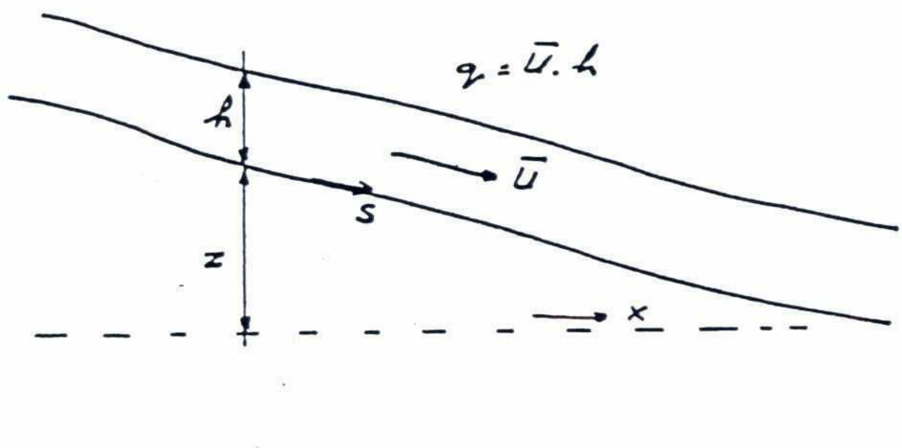

Fig. 6.1 Definition sketch

momentum water : $$\frac{\partial \overline{U}}{\partial t} + \overline{U}\frac{\partial \overline{U}}{\partial x} + g\frac{\partial h}{\partial x} + g\frac{\partial z}{\partial x} = -g\frac{\overline{U}|\overline{U}|}{C^2 h} \quad (6.1)$$

continuity water : $$\frac{\partial h}{\partial t} + \overline{U}\frac{\partial h}{\partial x} + h\frac{\partial \overline{U}}{\partial x} = 0 \quad (6.2)$$

transport sediment : $$s = s\,(\overline{U},\ \rho_s,\ D,\ C \text{ etc.}) \quad (6.3)$$

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PLANFORM CHANGES OF A BRAIDED RIVER WITH FINE SAND AS BED AND BANK MATERIAL

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ABSTRACT: As a first step towards the development of a predictive model for channel changes and related bank erosion, morphological processes were studied in a braided river with fine sand as bank and bed material. This was done using mainly (geometrically corrected) satellite images, the use of which was justified because of the large scale of the river studied: the Jamuna River in Bangladesh with a total width of up to 17 km. The study concentrated on bank erosion, channel shifts, and processes at bifurcations and confluences. It was found that the bank erosion depends on the relative curvature of the curved channels, and that the rate is an order of magnitude larger than predicted by Hickin & Nanson (1984). Several types of channel shifts were identified, and it was observed that cutoffs occur already at very low cutoff ratio's. The main conclusions from the study are that on the one hand much more studies are needed to improve the understanding of these different processes, and on the other hand that there is a clear limit to the period over which predictions can be made due to the observed chaotic behaviour of the channel changes.

1 INTRODUCTION

Understanding of future planform changes of rivers is essential for the siting of infrastructure (like embankments, supply canals, roads, etc.) and of towns and villages along rivers. Nowadays the understanding of planform changes of meandering rivers is fair: meandering via bank erosion can be predicted to some extent (Hickin & Nanson, 1984), including the occurrence of cutoffs (Klaassen & van Zanten, 1989), and predictive models for the development of meanders (Parker & Andrews, 1981; Crosato, 1990). No predictive models are available for braided river systems, although the need for predictions of future behaviour is even more accute, because of the apparently erratic behaviour of such river systems (Burger et al, 1988).

This paper reports on a study that was carried out to improve the understanding of the processes in braided river systems, with the ultimate aim to develop a deterministic method for predicting the future changes in channel planform and the corresponding bank erosion. Connected to this is the question whether there is a limitation as to the period over which bank erosion along a braided river can be predicted in advance. The present study was initiated in relation to the design of a bridge across the braided Jamuna River in Bangladesh. The Jamuna River is the lowest reach of the Brahmaputra River in Bangladesh (see Fig.1). Its main characteristics are a water level slope that gradually decreases form 0.10 to 0.06 m/km, the bed material is quite uniform and D_{50} varies from 0.25 mm near the Indian border to 0.16 mm at the confluence with the Ganges River, the average annual flood is about 60,000 m3/s, and during low flow between 4.000 and 12.000 m^3/s. The Jamuna River is a large braided sand bed river, the number of braids (during low flows) varies between 2 to 3, and the total width of the braided channel pattern varies between 5 and 17 km. Flood conditions usually prevail from May through October (see e.g. Coleman (1969)) while the low flow season lasts from December through March. For more details on the Jamuna River reference is made to Coleman (1969), Bristow (1985), Klaassen & Vermeer (1988a and 1988b) and RPT et al (1987, 1990a and 1990b).

Within the Jamuna Bridge Appraisal Study, an extensive analysis was made of rates of bank erosion along and channel processes in the Jamuna River in Bangladesh, using satellite images and cross-sections.

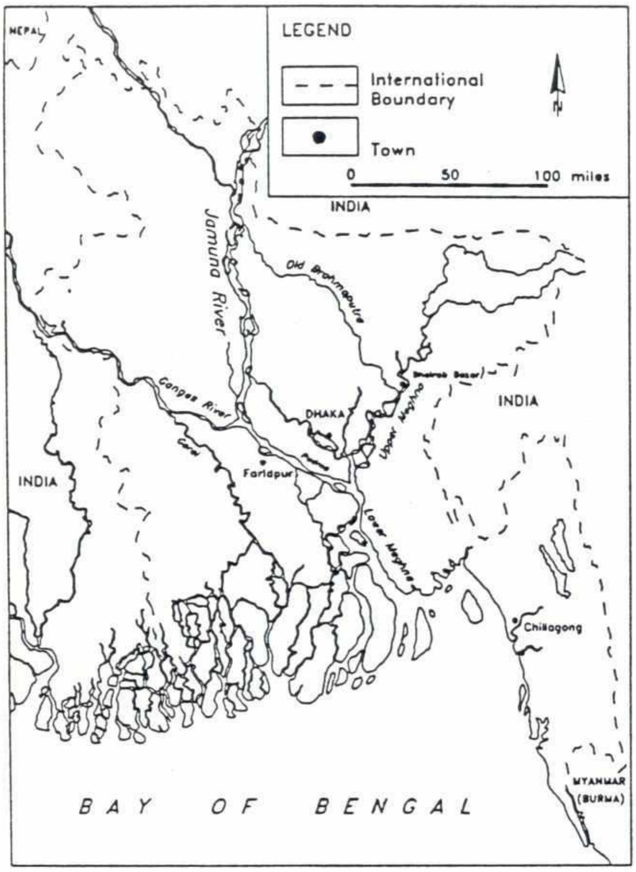

Fig. 1 Map of Bangladesh with Jamuna River

This paper summarizes the results of this study. For more details reference is made to Masselink (1989).

Several studies on the Jamuna River have already been carried out, leading to an increased understanding of the processes that take place in such large, braided sand-bed rivers (Coleman (1969), Bristow (1985), Klaassen & Vermeer (1988a,b), Klaassen et al (1988)). These studies have revealed processes that take place on a scale which can not be compared to the results of studies on "normal" braided rivers (see e.g. Rust, 1972). In particular the use of satellite images in the present study does enhance the understanding of the processes in the Jamuna River substantially. Because of the large scale of the river involved the accuracy of the LANDSAT MSS images, in combination with the rapid and substantial changes, makes the use of the satellite images very fruitfull. For the time being, SPOT images are not a real alternative as (i) they cover a much shorter period and (ii) the detailed insight they provide, is not really needed in this stage.

2 APPROACH

2.1 Data used

Yearly bank erosion rates were studied on the basis of comparisons of geometrically and otherwise corrected satellite images. Here LANDSAT MSS images of in total 6 years were used, with time intervals of 1, 2 and 6 years. The data that were used in this study comprised cross-sectional data and planform data derived from LANDSAT images.

2.2 Cross-sectional data

The cross-sectional characteristics of the Jamuna River were studied on the basis of soundings made by the Bangladesh Water Development Board (BWDB) since 1966. The locations of the studied cross-sections are indicated in Fig. 2.

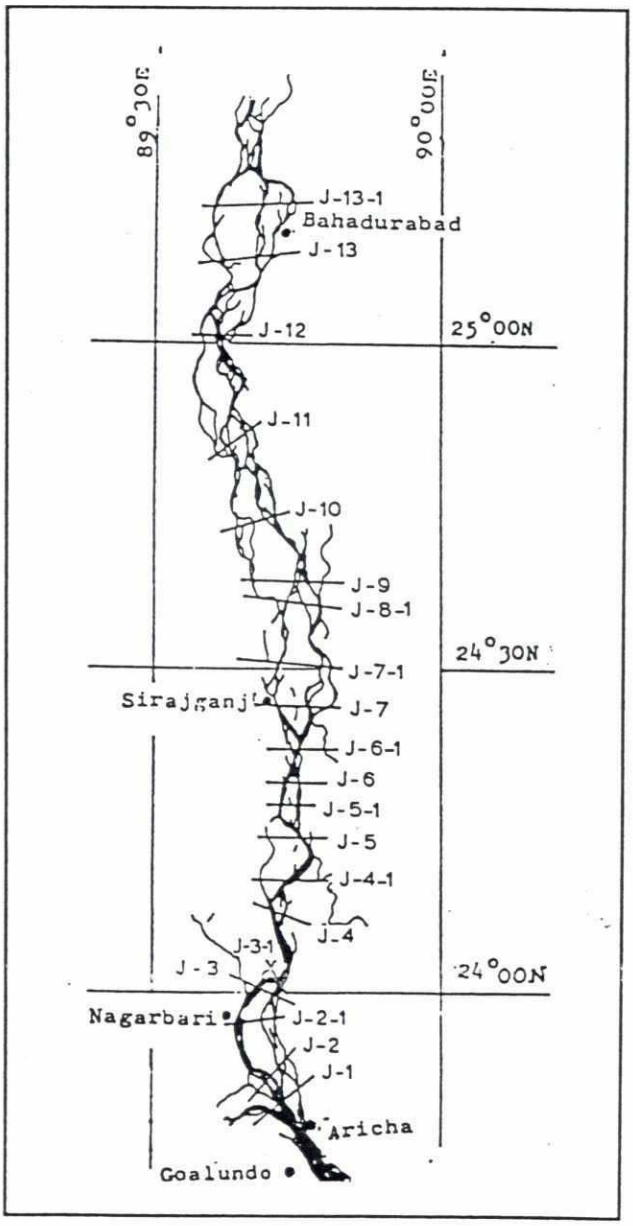

Fig. 2 Location of BWDB cross-sections

An extensive analysis of all cross-sections has been carried by Klaassen & Vermeer (1988a). They concluded that over the last decades the Jamuna channel bed was stable both laterally and vertically. Hence, the Jamuna River is on the average not aggrading, and can be supposed to be in equilibrium.

2.3 Selected satellite images

For the LANDSAT MSS images selected see Table 1.

Image Scene number[1])	South 138-43	North 138-42
Year	Date	Date
1976	4 Mar	-
1977	9 Feb	9 Feb
1978	22 Feb	-
1984	23 Feb	23 Feb
1986	20 Feb	20 Feb
1987	7 Feb	7 Feb

[1]) LANDSAT 4/5 numbering

Table 1 LANDSAT images used for the present study

The research was primarily based on the images of the Southern part of the Jamuna, because these images form a more or less continuous series, whereas the Northern images did not. Only images were selected for dates for which the stage at Sirajganj (see Fig.2) was between 6.00 and 7.00 m+PWD (PWD refers to the agreed reference level in Bangladesh). Hence, all images show the Jamuna River during low flow conditions. The images were geometrically corrected (de Jong, 1988), hence mutual comparisons between different years could be made.

2.4 Classification

In a next step the images were classified according to the type of soil and the vegetation cover. For this an algorithm was applied, using two spectral bands. For the procedure applied see Klaassen et al (1988) and de Jong (1988). The following classification and corresponding colour coding was adopted:

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

The classified images are called statical composites. An example of a series of these statical composites is given in Fig. 3. Because it concerns a black and white copy, here the water corresponds to the darkest colour. Changes over the years in water and sand distributions are obtained by comparing statical composites. An "overlay" (of the digital data) of two statical composites is made, and based on the four identified classes, there are 16 possible combinations, yielding 16 new classes. These can subsequently be interpreted and analyzed. The new picture obtained (being the comparison of two succesive years) is called a dynamical composite.

2.5 Processing, analysis and interpretation

The statical and the dynamical composites were used, together with the BWDB cross-sections, for the study of processes active in this braided river system. The following topics were investigated:
(1) bank erosion rates of curved channels,
(2) cutoffs,
(3) processes at bifurcations,
(4) processes at confluences.
The rationale for selecting these topics is that it is felt that these processes in combination are responsible for the changes in channel patterns. The results of the various analyses are presented in the subsequent Chapters 3 through 6.

The different processes were studied via measurements in the statical and dynamical composites. The accuracy of the LANDSAT-image after processing is approximately 200 meters at a significance level of 97%. De Jong (1988) presents a complete discussion on the selection, processing, classification and accuracy of the images.

3 BANK EROSION RATES

3.1 Bank erosion rates

Using the dynamical composites, the bank erosion rates along curved channels could be studied. Not all curved channels were included in the analysis: channels that had significantly narrowed or widened in one or two years were excluded, as for these cases it was difficult to distinguish between bank line changes due to an increase or decrease of discharge and 'true' bank erosion. The bank erosion does obviously vary along the bend: here only the maximum bank erosion rates were studied.

Fig. 4 provides a summary of the observations for the four dynamical composites of the south image, via a plot of the observed erosion rates (divided in classes) versus the number of observations in each class. Yearly erosion rates E apparently vary between 0 and 1,000 m. Fig. 5 provides the same information, but here the average number of occurrences for the four different periods is presented. It appears that the bank erosion rate along the curved channels of the Jamuna River in most of the cases is between 0 and 500 m/year with larger values up to 1,000 m/year under exceptional conditions. This is conform the findings of Coleman (1969).

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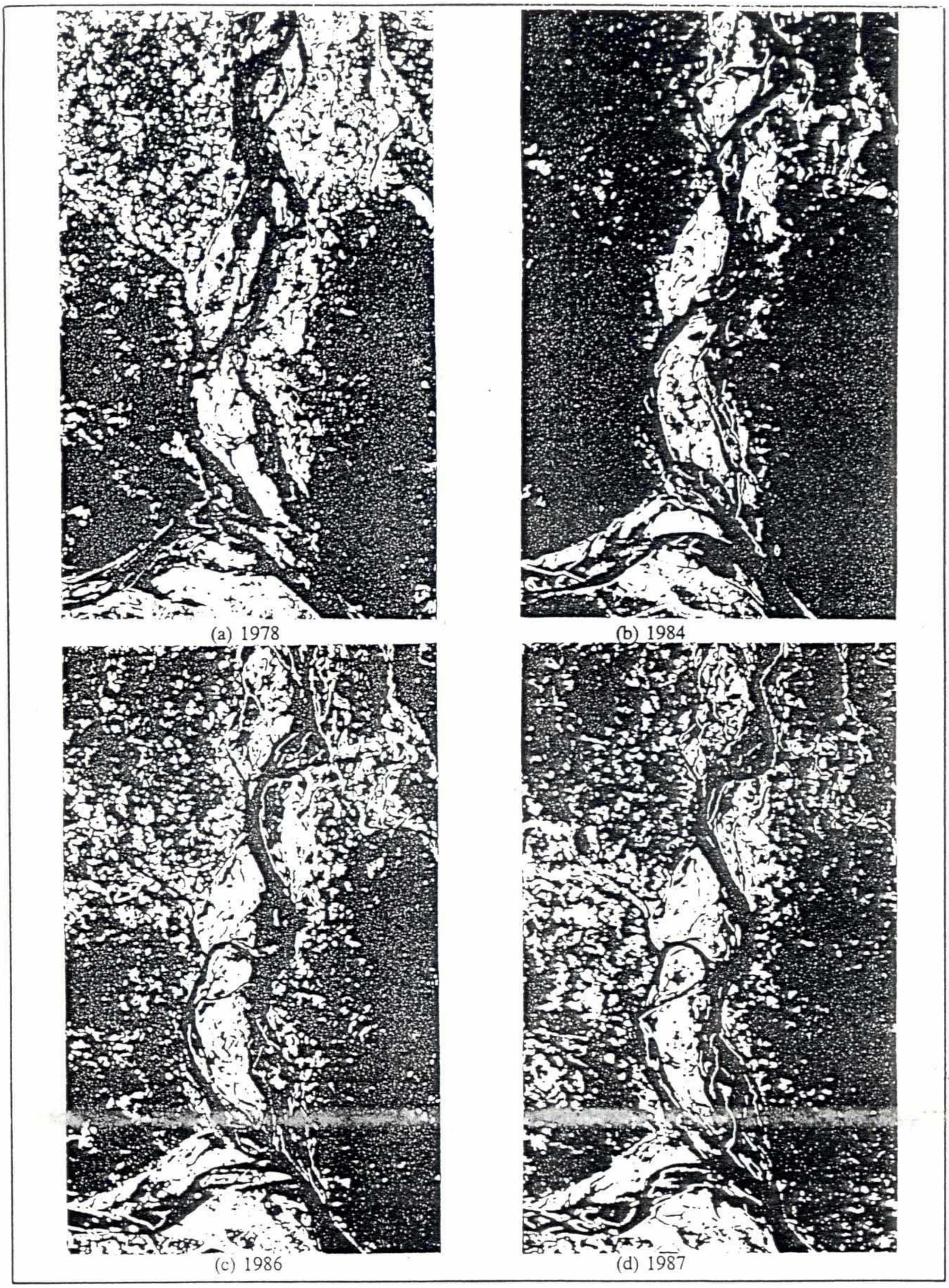

Fig. 3 Classified LANDSAT images of the Southern part of the Jamuna River for four years (approximate scale 1:500,000)

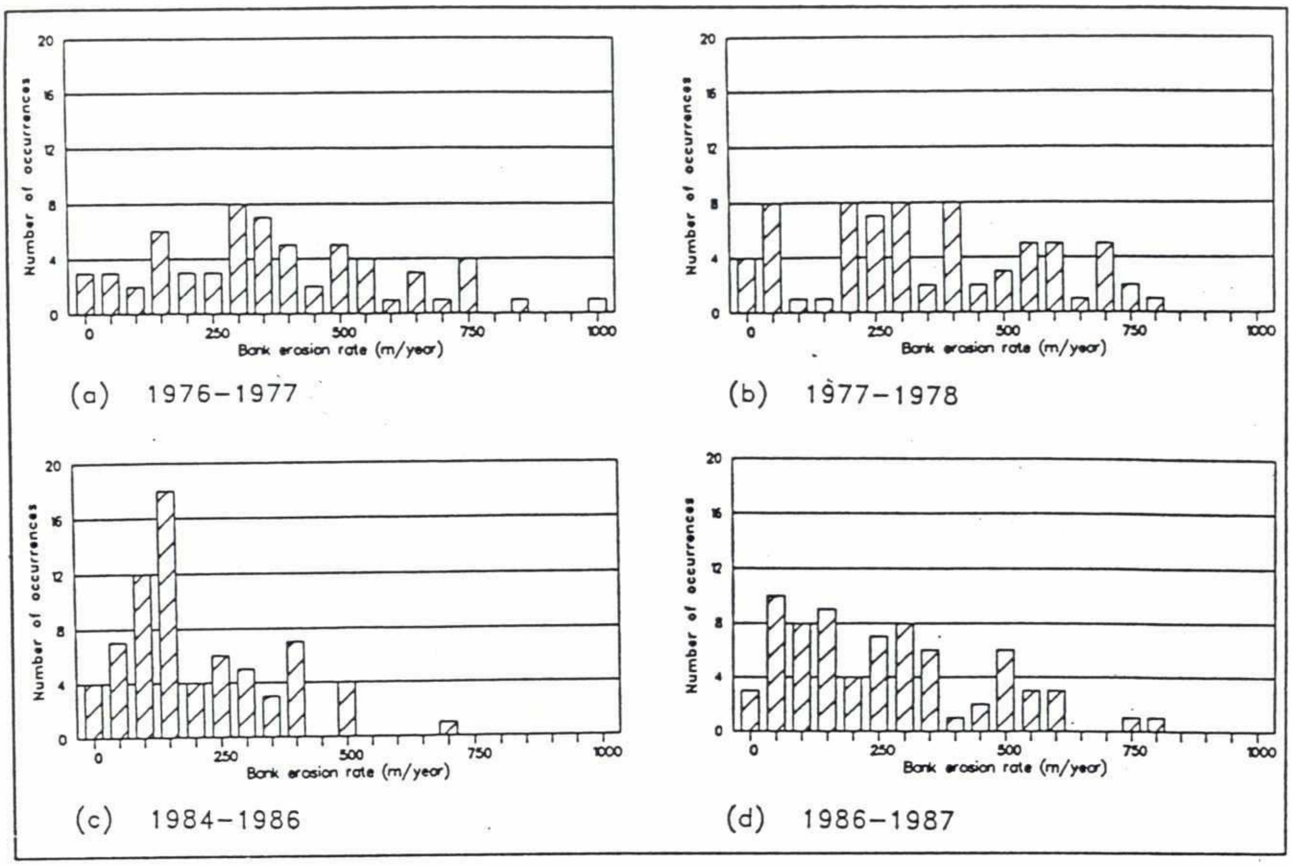

Fig. 4 Observed bank erosion rates along curved channels of the Jamuna River

Also some other aspects of the bank erosion were studied using the satellite images. The direction of the bank erosion was studied by plotting in Fig.6 the direction of the maximum erosion relative to the valley slope. The direction of the largest erosion is on the average approximately perpendicular to the valley slope, while for smaller erosion rates it frequently deviates substantiaaly from the valley slope.

Also the influence of vegetation was studied. It was found that chars with minor or absent vegetation do not erode faster than vegetated floodplains along the

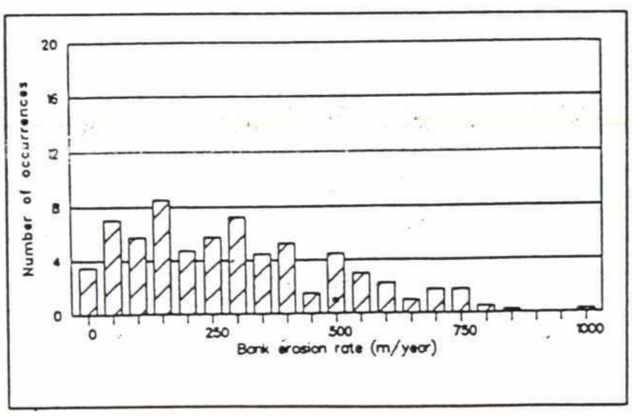

Fig.5 Average bank erosion rates along curved channels of the Jamuna River for the four periods considered in Fig. 4

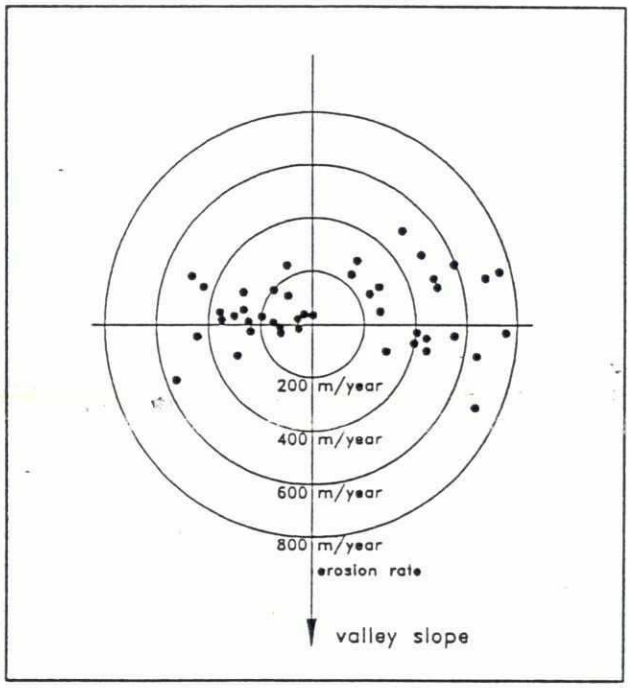

Fig. 6 Direction of bank erosion along curved channels of the Jamuna River

edge of the channel pattern. Hence it can be concluded that for the deep channels of the Jamuna River (see Klaassen & Vermeer, 1988b) the influence of vegetation only effective in the upper layers is negligible.

Furthermore a distiction was made between the following bank erosion mechanisms (see the below Fig. 7): rotation (A), extension (B) and translation (C).

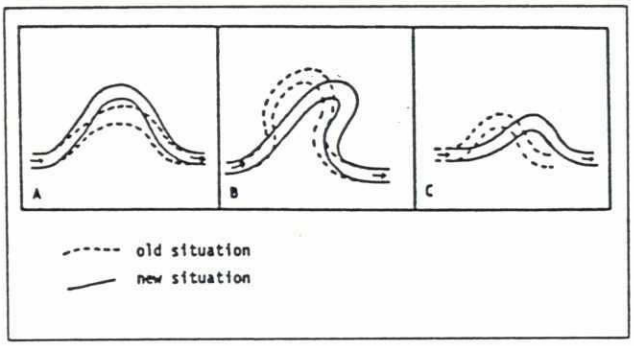

Fig. 7 Possible bank erosion mechanisms studied here

It was found that both rotation and extension do occur; translation is absent along the Jamuna braids. Translation does usually occur for cohesive banks. The chars and flood plain deposits of the Jamuna River exhibit hardly any cohesion, so this may be an explanation.

Finally it is remarked that apparently the yearly flood hydrograph may play a role in the magnitude of the yearly erosion rate (see the difference between e.g. 1976-1977 and 1984-1986). This has not been explored in more detail yet.

3.2 Analysis

The observed bank erosion rates were analysed within a frame-work similar to previous work by Nanson & Hickin (1985) for meandering rivers, by establishing the relation between the bank erosion rates and the relative curvature of the channels. Here a complicating factor is the fact that not one channel is considered, but that several parallel braids are present. In a meandering river the bankfull width is fairly constant (and fairly good related to the 'dominant' discharge). In the present case the width of a channel will differ depending on whether the channel is in regime, scouring or vanishing. This makes scaling with the width W as parameter (as done by Nanson & Hickin) more risky. Nevertheless this has been attempted hereafter.

According to Nanson & Hickin the yearly bank erosion E (in m) can be expressed as:

$$E = f(W, R/W, C, B) \qquad (1)$$

where E = yearly bank erosion (m), W = channel width (m), R = radius of curvature of the curved channel (m), C = Chezy coefficient ($m^{1/2}/s$), that represent the channel roughness, and B = overall bank resistance coefficient. For the time being it was assumed that the parameters C and B do not vary along the Jamuna River, hence for this river the equation would reduce to :

$$E = f(W, R/W) \qquad (2)$$

This expression was tested for the Jamuna bank erosion data. The determination of the applied radius of curvature was done in the following way. Usually the values of relative curvatures (R/W) for the two years were averaged. If, however, one of the values for R/W was smaller than 5.0, and the other was greater than 5.0, then the value of the bend with the smallest relative curvature was used. The reasoning behind this choice is that it was considered that this bend was most active in the eroding process.

Initially a distiction was made as to the channel width: four classes were identified (see Fig. 8). The results, related to bends selected according to the criteria given in Section 3.1, are presented in Fig. 8. It can be observed that the largest channels tend to correspond to the largest erosion rates. Hence a similarity collapse was attempted, by plotting E/W versus R/W. The result is given in Fig. 9.

Upon inspection of Fig. 8 and Fig. 9 the following observations can be made:

(1) There is definitely a negative correlation between the relative bend curvature (R/W) and the erosion rate (E/W). Low relative curvatures lead to relatively fast erosion rates, and vice versa. There is no tendency for very sharp bends ($R/W < 2.5$) to have smaller erosion rates, as was observed for meandering channels by Nanson & Hickin (1984). The value of W used here corresponds to the low flow width. As on the average the bankfull width is about 3 times larger (Klaassen & Vermeer, 1988a), a possible peak would have occurred here for the low flow data for R/W-values of approximately 7.5.

(2) The largest bends ($W > 1000$ m) plot low in Fig. 9, indicating that relative erosion rates by large channels is relatively small. However, only 7 large channel bends were studied. From Fig. 8 it appears that there is no significant difference in relative erosion rate between the smallest ($W < 500$ m) and the intermediate ($500 < W < 1000$ m) channels.

(3) Both Fig. 8 and Fig. 9 show a substantial scatter. Apart form the assumptions made during the

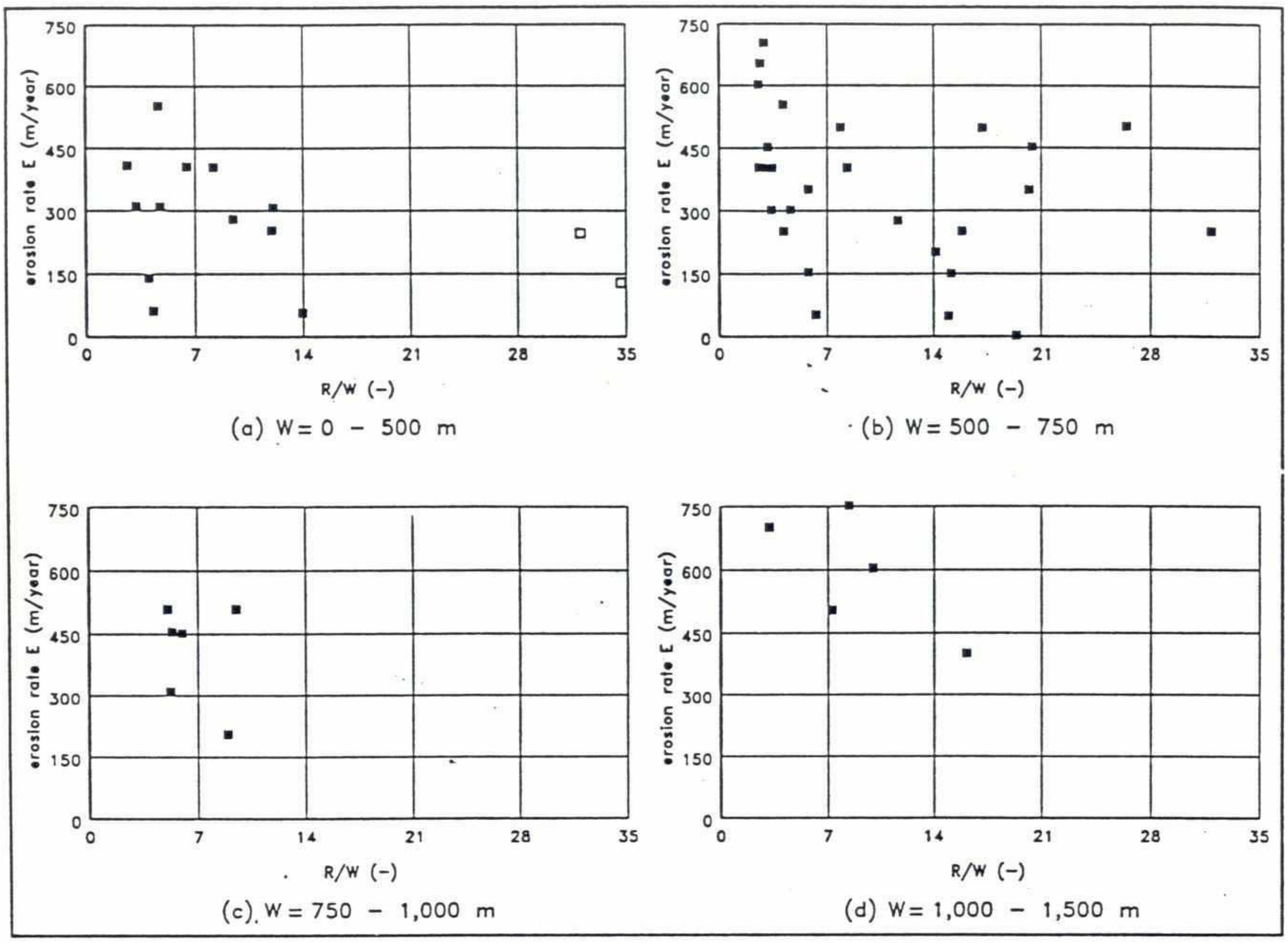

Fig. 8 Dimensionless erosion rate E/W versus R/W for the channels classified according to width

derivation of Eq. (2), possible explanations for this scatter may be:

- measuring accuracy (see before);
- use of low flow widths;
- use of one value of the radius of curvature, where in reality R may vary considerable along the bend;
- the inclusion of less active bends possibly being rather shallow.

A comparison was made between the presently observed erosion rates and the rates predicted byHickin & Nanson (1984). Assuming a bankfull discharge of a channel of 20,000 m^3/s and using the other river characteristics described in Chapter 1, the estimate of several tens of meter of erosion a year. It is clear that this estimate is an order of magnitude smaller than what was actually observed. A possible

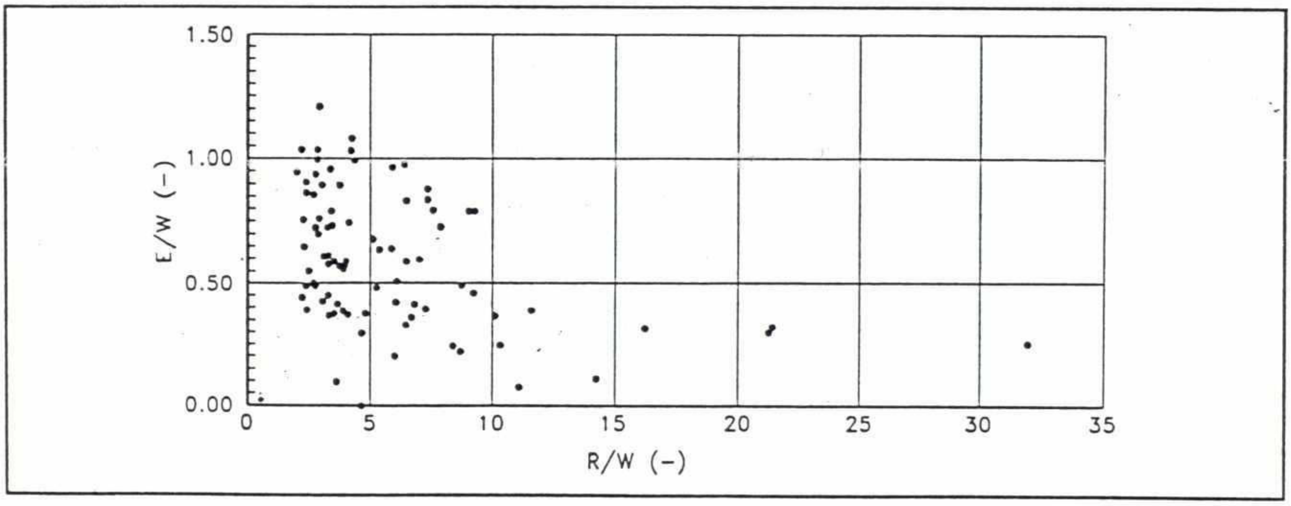

Fig. 9 Dimensionless erosion rate E/W versus R/W for all curved channels along the Jamuna River

explanation could be that in the case of the Jamuna River, the floods extend over a period of some three months. It may be appropriate to include in any prediction method also the duration (and magnitude?) of the floods.

4 CHANNEL SHIFTS

Using the dynamical composites, also channel shifts were studied. The channel pattern of the Jamuna River changes continuously: large channels being abandonned, and new channels developing in a few years only, are common features. Coleman (1969) refers to this process as to 'sudden shifts'. A channel shift is accomplished by the development of a completely new channel, or, more common, a pre-existing channel takes over the conveyance function of another channel. The new channel may flow through the original channel deposits or through the floodplain. Three types of channel shifts were identified and these are discussed hereafter:

(1) Bar induced shifting

Sand bar induced channel shifting is not often identified in the literature, but is a rather important process in the Jamuna River. Large sand bars migrating in a channel or developing in specific channel reaches block the entrance of small channels. Consequently these channels will receive less discharge and are "abandonned" subsequently. Sand bars can also redistribute the flow so that one channel receives more discharge than another. A complete channel shift can be the result (see Fig. 10).

(2) Development of a cutoff

The development of a cutoff occurs frequently. Of the 23 cases of channel shifts studied, 11 relate to cutoffs. To get some insight into this phenomena, it was approached in two ways, notably (i) considering the relative curvature R/W, and (ii) considering the cutoff ratio (Klaassen & van Zanten, 1989).

Re (i) Relative curvature

The relative curvature of a bend is assumed to be a primary control on the processes which determine channel shifts of this type. From a study of channel shifts, it appeared that the average relative curvature (R/W) of the pre-shift bends was 3.6 (standard deviation 1.8). Bends with R/W-values < 3.6 are apperently 'unstable' and result in a channel shift within a period of two years. This may be explained by the shear stress distribution in the bend.

Re (ii) Cutoff ratio

Klaassen & van Zanten (1989) have shown the importance of the cutoff ratio λ, being the ratio between the length of the channel along the curved reach and the direct line, along which the cutoff channel is developing. For a number of cutoffs in the Jamuna River the value of λ was determined and a cumulative frequency distribution was prepared. The

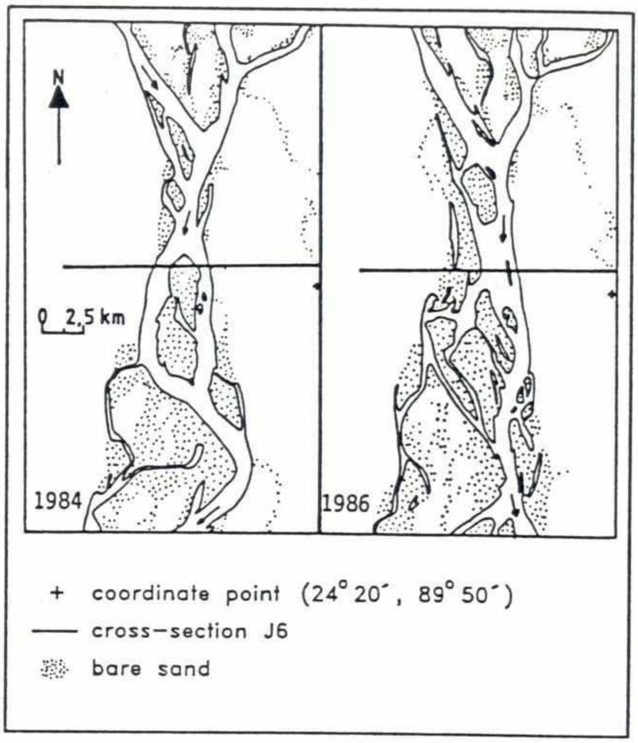

Fig. 10 Bar-induced channel shift

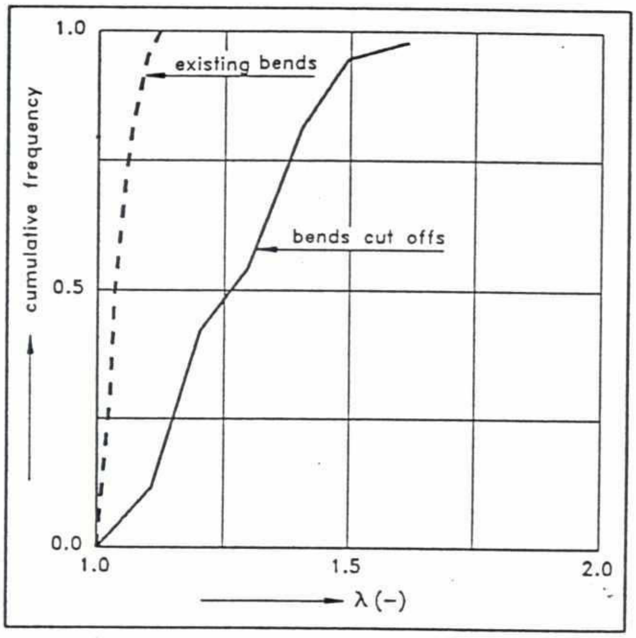

Fig. 11 Cumulative frequency distribution of cutoffs compared with existing bends

results are presented in Fig. 11. It appears that the values of λ vary between 1.0 and 1.7. These are very low values for the cutoff ratio, demonstrating that in this type of braided river with fine bed material channel shifting via cutoffs occurs relatively quickly. For comparison: in meandering rivers values of λ in the range of 5 to 30 have been observed (Joglekar, 1970; Klaassen & van Zanten, 1989). Furthermore it is observed that the 'critical' value of λ varies between a wide range, hence the usefullness of Figure 11 for predictive purposes is very limited. For the time being a 50% value of 1.25 can be used.

In Fig. 11 also the frequency distribution of potential cutoff ratio's of existing bends is added, and it is observed that larger (potential) values of λ apparently result almost immediately into a cutoff.

(3) Outerbend channels in bends

Also the formation of what is called here an outer bend overflow channel occurs frequenly (9 times out of the 23 cases studied). Also this may be related to the shear stress distribution in the bend (Masselink, 1989). Often such an outer bend overflow channel developes where previously an old channel has been present. See Fig. 12 for an example of the development of an outer bend channel, where a fairly rapid shift took place. The new channel started as an outerbend overflow channel and was located in the floodplain. Within two years (1979-1981) the overflow channel deepened from 7 to 17 metres, and the original channel had been filled in completely. The original channel is not always filled in completely, but may still convey a certain amount of discharge.

The previously discussed types of channel shifts were rather simple in that a clear cause and effect relationship could be determined: e.g. a sand bar blocks a channel, and consequently the channel is abandonned. Many channel shifts occur without such an obvious reason. Usually sand bar formation enhances a channel shift, but it is not the direct cause. It is proposed that changes in flow and sediment distribution cause local bed aggradation in one channel. Consequently the flow efficiency of this channel is reduced through a decrease in channel slope. Sand bar formation may occur. Another channel may take over the function of the aggrading channel because of its higher efficiency. All depends on the complex interaction between channel morphologhy, discharge and sediment load. This relationship is too complex to be studied on the basis of LANDSAT images alone. Complex channel shifts are a fairly important process in the Jamuna River, the reasons why such shifts take place however, are not fully understood.

5 PROCESSES AT BIFURCATIONS

When a major channel continues in two smaller channels, a bifurcation is present. A bifurcation is a very important feature in a braided river system, as the development in time of the two bifurcating channels is determined by the flow and sediment distribution at the bifurcation. Here a description is given of the processes that are active at a bifurcation. The two channels arising from the bifurcation are

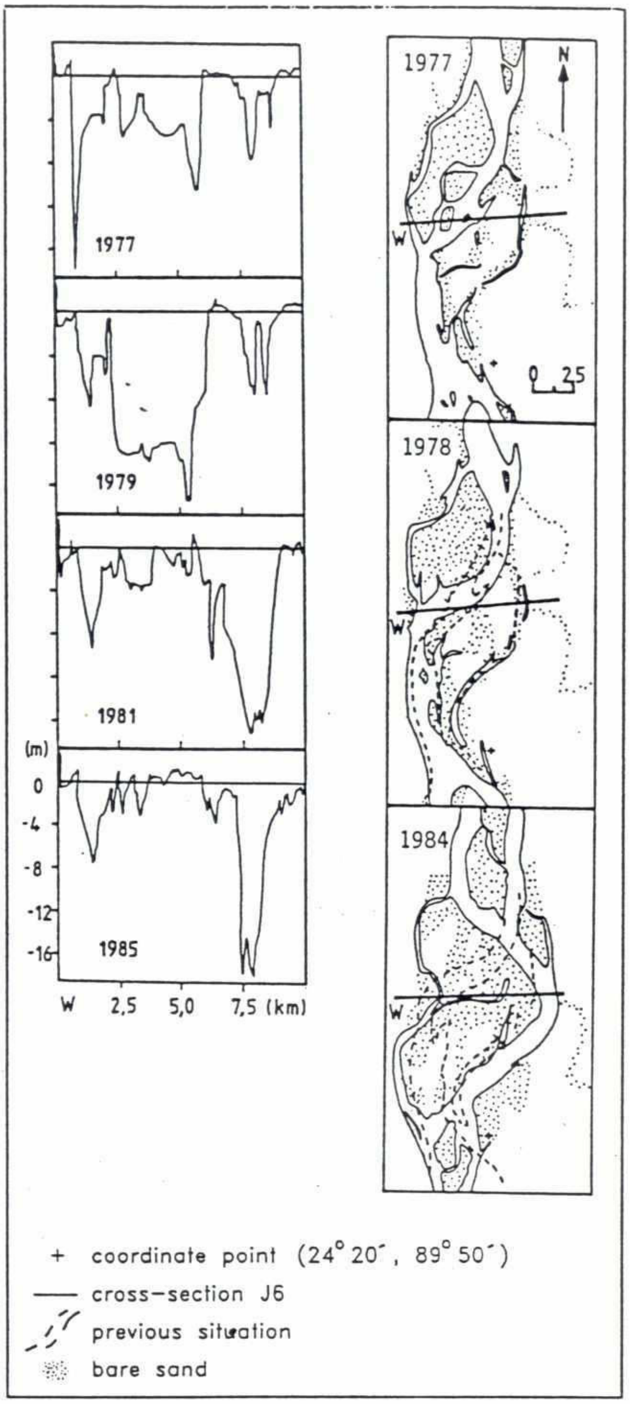

Fig.12 Development of an outer bend channel into an already existing old channel

separated from each other by a char; the surface of this char is above bankful discharge level.

The development of the bifurcation depends on the character of the planform; it may be a symmetric or an a-symmetric one. A symmetric bifurcation is characterized by the flow direction of the upstream channel being different from the two downstream channels. A symmetric bifurcation is characterized by upstream accretion. An analysis of several large bifurcations revealed an average propragation speed (in upstream direction) of approximately 900 m/year.

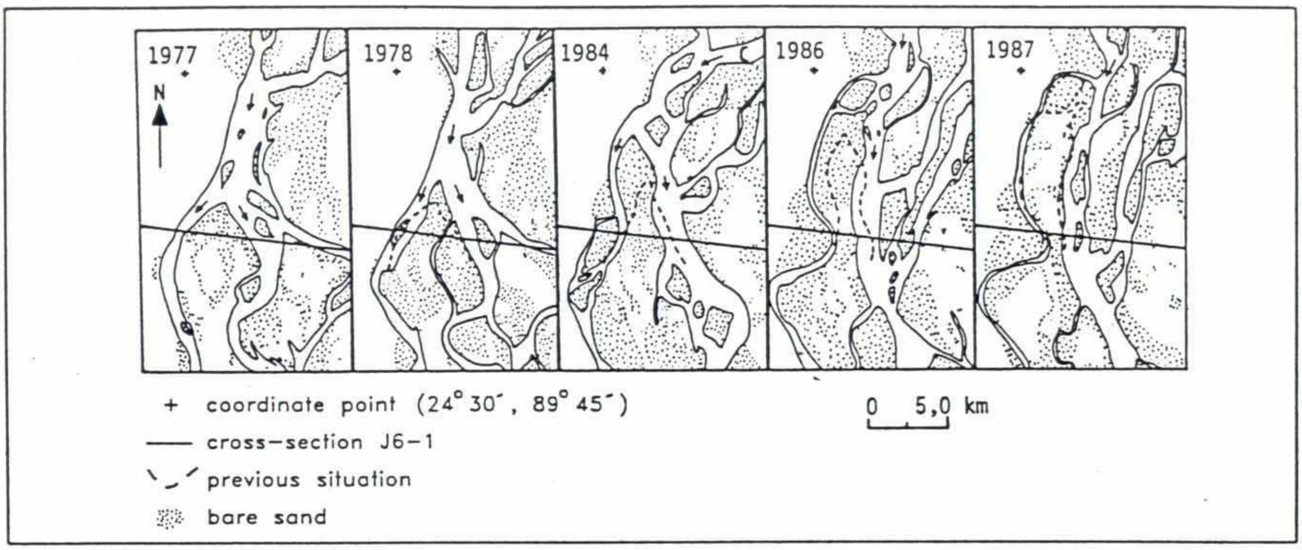

Fig. 13 Development of a bifurcation during the period 1977-1987

(in upstream direction) of approximately 900 m/year. It appears that the propagation rate scales with the size of the channels.

An a-symmetric bifurcation is characterized by one of the downstream channels having approximately thesame direction as the upstream channel. The other downstream channel is usually much smaller in size,and because of blocking of this smaller channel by chars, it usually disappears in one or two years.

Fig. 13 outlines the development of a bifurcation in the period 1977-1987. The behaviour is characterized by a symmetric and an a-symmetric phase. In the period 1977-1984 a symmetric bifurcation is observed which progrades in an upstream direction at a rate of about 625 m/year. Between 1984 and 1986 a major channel upstream of the bifurcation (marked with a c on the drawing for 1984) is abandoned. The result is an asymmetric bifurcation where the eastern bifurcation channel is transporting most of the discharge. A small char is formed, blocking the entrance of the western channel. In 1987 the small char is attached to the major char and the bifurcation point has disappeared.

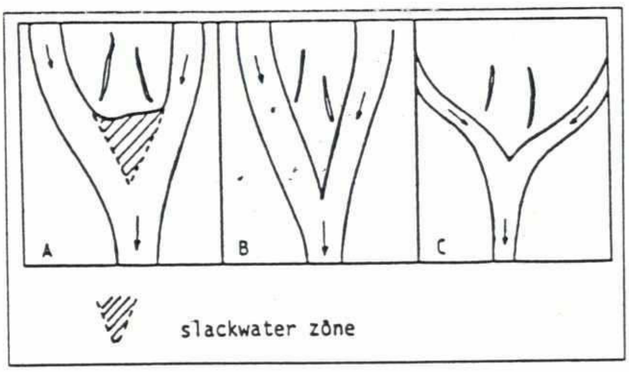

Fig. 14 Different types of confluences

6 PROCESSES AT CONFLUENCES

The development of a confluence appears to depend on its shape. Different shapes of confluences are schematically drawn in Figure 14. They are different in the presence of a slackwater zone (A versus B and C) and in the angle between the two confluencing channels (B versus C). A slackwater zone is prone to quick deposition and usually vanishes in one year. The channels that drain the char surface have an important role. The char surface slopes in downstream direction so most of these channels discharge near the confluence. If they discharge in the slackwater region all the sediment load is depositing there, giving a significant contribution to the downstream accretion. If the confluence is a smooth one, all the sediment will be discharged in the main channel and be transported downstream. In this case these channels may even lead to erosion of the confluence as most of their sediment load is extracted from the downstream part of the bar.

Bristow (1985) mentions downstream accretion is an important mechanism for the growth of medial chars (major bars in main channels). In the present study this was not confirmed for the major confluences studied here. In general if the confluence margin is smooth, and the two confluencing channels do not change significantly, confluences are relatively stable river sections. This corresponds to the conditions in the Yellow River, where some of the nodal points are formed by channel confluences (Chien, 1961). If on the other hand one of the channels becomes dominant, the confluence moves in the direction of the minor channel by means of erosion at the major channel side of the confluence, and sedimentation at the minor channel side. On the average, confluence points do not move significantly upstream or downstream.

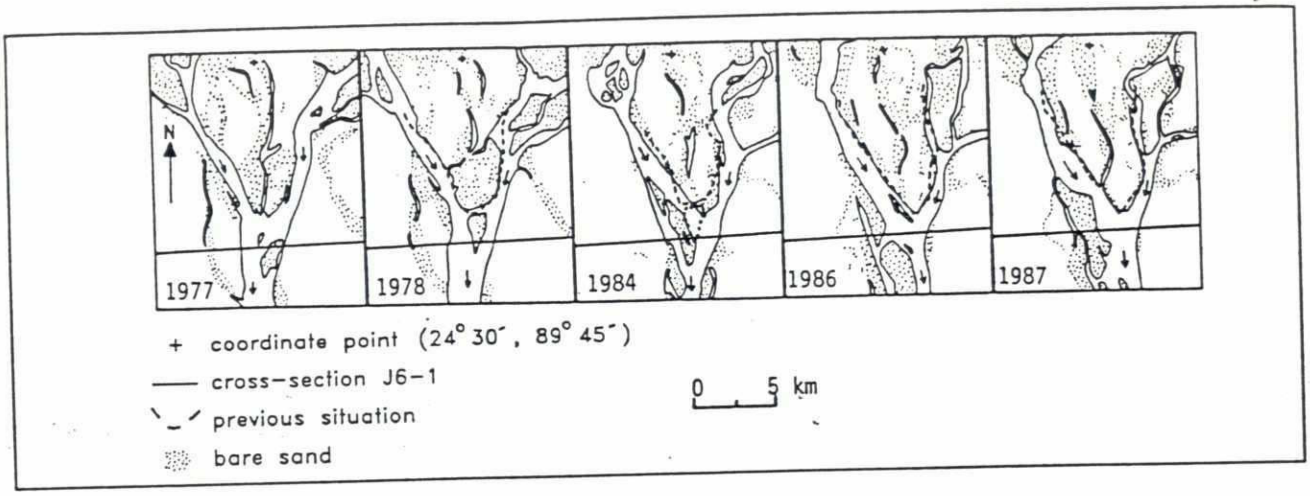

Fig. 15 Development of a confluence in the period 1977-1987

As an example here the historical development of a confluence near Sirajganj over the period 1977 -1987 is shown Fig. 15. This confluence had a fairly blunt shape in 1977. In 1978 a bar had been formed by sedimentation in the slackwater region, smoothing the confluence. Over the period 1978-1984 the western channel became more important, and the conflunce migrated to the east. In the period 1984-1987 the situation did not change significantly, and over the whole period the position of the confluence is remarkable stable. Old maps, however, show that the conflunce was not present at this particular location in the 19th century. Hence, this so-called nodal point (Coleman, 1969) is probably more determined by the presence of a confluence in the last decades than by other possible causes, like larger resistance to erosion due to deposits of a former river (Coleman, 1969).

7 TOWARDS A PREDICTIVE MODEL?

The ultimate objective of this study was to investigate whether the development of a predictive model for braided rivers with fine bed and bank material is feasible. With such a deterministic model one should be able to predict future channel development and related bank erosion with reasonable accuracy and preferably on a time scale of at least several years.

From the results of the study reported here it can be concluded that presently it is not possible to develop such a deterministic model, for at least two reasons:

(1) the limited understanding of the prevailing processes, and

(2) the observed chaotic behaviour of the system.

These two reasons are discussed hereafter.

Re (1) Limited understanding

Even for the limiting assumptions made here (e.g. in the analysis of the bank erosion rates that the channel should not 'deteriorate' too much over the period of comparison), the resulting relationships exhibit a large scatter. This is a.o. caused by not including all relevant features in the analysis. Some examples of this are:

(1) The analysis was generally based on the LANDSAT images from which only two-dimensional features can be studied. The BWDB cross-sections (taken only every 4 km) were only of limited help. Especially channel shifts are strongly influenced by the surface morphology. Relatively low parts of the floodplain are prefered locations for new channels, and generaly such locations cannot be determined from LANDSAT images.

(2) The studied images form a discontinuous series (1976 (only partly usable, because of cloud cover), 1977, 1978, 1984, 1986, 1987). Consequently the development of a bend or a secondary channel could only be "followed" for two succesive years. After the 'gap' that then followed, usually the bend or secondary channel did not exist anymore.

(3) The shape of the discharge hydrograph was not yet taken into account in the analysis of e.g the bank erosion rates. The same holds for channel shifts: Klaassen & van Zanten (1989) have shown theoretically that the magnitude of the flood and its duration) are important parameters in the actual occurence of a cutoff.

Re (2) Chaotic behaviour

Morphological processes in braided rivers like the Jamuna River ('choked with sediment', because of the fineness of the bed material and the large sediment loads during floods), appear to be characterized by chaotic behaviour, and this makes it difficult if not impossible to model the morphological processes in a deterministic way. A.o. sand bar induced channel shifting is producing this chaotic behaviour, but also the discharge hydrograph being different for different years adds to the chaotic behaviour.

The results of the present study imply that for the time being predictions can be made, but only for the conditions one or two years ahead. The only locations for which predictions can be made for a longer period ahead are the nodal points (major confluences), which appeared to be quite stable over decades.

8 CONCLUSIONS AND RECOMMENDATIONS

Based on the study reported here, the following conclusions can be drawn:

(1) The study of the satellite images, in combination with cross-sections has provided an improved understanding of changes in channel pattern of and the related bank erosion along large braided rivers with fine sand as bank and bed material.

(2) This improved understanding relates especially to the relation between the erosion and the relative curvature (R/W), the negligible importance of vegetation on bank stability, the occurrence of channel shifts and the importance of sand bars, the propagation in upstream direction of bifurcation when the two channels are of almost equal importance, and the stability of major confluences.

(3) Nevertheless, the understanding of these processes is still very limited, and does not allow for the development of a deterministic model for the development of the channel pattern and the prediction of where bank erosion will take place and how large the bank erosion will. Only near confluences prediction over some years may be possible, because usually these reaches are fairly stable.

(4) The fact that a deterministic model cannot be developed, is not only due to the as yet too limited insight in the processes in these rivers, but probably also to chaotic behaviour.

Based on these conclusions, the following recommendations are made:

(1) to study the morphological processes in more detail by:
- including also information over channel dimensions, in particular the relative importance of eroding channels,
- studying a more continuous series of satellite images,
- to include other aspects in the analysis like the duration of the yearly hydrograph;

(2) to carry out a special study into the possible chaotic behaviour of the river system, with the aim to identify this chaotic behaviour and to study its implications for the time span over which a reliable prediction of channel changes and bank erosion can be made.

9 ACKNOWLEDGEMENTS

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LIST OF SYMBOLS

B = bank erosion coefficient
C = Chezy coefficient ($m^{1/2}/s$)
E = yearly erosion (m)
R = radius of curvature (m)
W = channel width (m)
λ = cutoff ratio (-)

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