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GROYNES AND TRAINING WORKS AFFECTING RIVER PLANFORM
A literature review

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ABSTRACT

Engineers try to control the flow conditions, bed levels and planform of rivers using a range of structures collectively known as river training works. Groynes are structures which are used for a variety of river training purposes. This report reviews published information on the physical response of rivers to the construction of training works in general and groynes in particular. It also includes a review of information on general design parameters for groynes such as spacing, orientation, plan shape etc. Recommendations are made concerning the development of computer models to predict the morphological impact of training works and also on the need for fundamental research into riverbank erosion which must be included in a computer model of planform response.

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1. INTRODUCTION

Rivers play a complex role in the geomorphological processes of land formation and erosion. During low flows, or as floods recede, a river can deposit sediment and silt up an important navigation channel. At high flows a river can erode banks and threaten property. River training works aim to control these natural processes and "train" the river to behave as men require. Engineers attempt to control the flow conditions, bed elevation and planform of a river using a range of structures collectively known as river training works. River training works encompass a wide range of engineering works, from stone groynes for channelising a braided river to transplanted vegetation for stabilising an eroding bank.

Chang (1988) lists the principles of river training as:

(1) training works must be designed to resist the design flow; and
(2) the impacts on the river should be understood and evaluated wherever feasible.

Any plan for river training works must therefore be evaluated with regard to channel response. This report aims to review what is known about the design of certain river training works and what is the probable response of the river to their construction.

The term "river training works" is so all embracing that it is necessary at the outset to limit the scope of this review. Because of the complex and inter-related nature of morphological processes, any works on a river could affect planform. Grade control structures such as sills or flood protection works such as embankments could have an impact on planform, but this is not their primary objective.

In terms of their physical impacts, river training works can be considered as active or passive. Bank protection works such as revetments or vegetation are used to control planform by stabilising existing banks to contain flow. These are referred to as passive works. Groynes and structures protruding from the bank will guide or redirect flow and can have a dramatic impact on planform. These are referred to as active works.

This report reviews what is known about the general design and impact of active river training works used to directly affect planform. By general design it means parameters like length, alignment and spacing rather than construction details such as the thickness of filter layers. Bank protection works are not reviewed in any depth. For a review of bank protection materials and methods see the recent book by Hemphill and Bramley (1989). The review covers published information in research reports, papers and design manuals. Based on this review, recommendations are made concerning research which could be undertaken to improve design and our ability to predict the impact of training works.

This review forms part of a research programme into two-dimensional morphological river modelling at Hydraulics Research (HR). For several years, HR has been using one-dimensional morphological river models to predict long-term bed level changes along a river, for instance downstream of a proposed dam. HR now proposes to develop a two-dimensional mobile-bed river model to predict changes in planform and bed level across and along a reach.

The terms of reference for this literature review were set down in 1988. Towards the end of writing this report the author came across two excellent textbooks by Brookes (1988) and Chang (1988) which cover a similar subject area. The present study complements these works.

2. BACKGROUND

2.1 Historical context

River training has a long history. The construction of flood embankments and bank protection works on such rivers as the Nile and Euphrates has been traced back thousands of years. In Europe, river training reached a high degree of sophistication in the 18th and 19th centuries with the channelisation of such rivers as the Rhine and Garonne. The early hydraulic engineers called themselves "hydrotects" with the idea that river engineering like architecture was more of an art than a science. The laws of alignment formulated by Fargue and the textbooks on "hydraulic architecture" by Belidor (1737) became standard references for a century. Figure 1 shows Belidor's theory of groynes as a measure against erosion based upon his observations and understanding of geometric principles. The drawings and design rules in these textbooks were based upon full-scale experimentation on major rivers, and Vischer (1987) suggests that they still contain lessons for today.

Early river engineers based their theories of the design and impact of training works upon experience partly because they lacked two vital equations: an open channel flow resistance formula; and a sediment transport formula. Even when these equations were developed their application remained impractical until the widespread availability of computers in the sixties. Before then, physical models and regime theory were the main tools used to study channel response for the last fifty years. But increasingly the equations of fluid flow and sediment transport are being combined into computer based morphological models which are used to predict the response of a river to proposed works.

2.2 Erosion, deposition and meander migration

The channel form of a natural river flowing through erodable sediments is a consequence of precipitation, drainage basin and sediment characteristics. A natural channel adjusts its cross-section and plan geometry by erosion and deposition of bed and bank sediments. A description of the different river bank erosion mechanisms is given by Keown et al (1977) and in the manual by Brandon (1989). Figure 2 shows the different erosion processes at a channel section. The mechanisms of bank erosion are:

a. Attack at the toe of the underwater slope, leading to bank failure and erosion. The greatest period of bank failure usually occurs at medium stage or lower.

b. Erosion of the soil along the bank caused by current action.

c. Sloughing of saturated cohesive banks due to rapid drawdown.

d. Flow slides (liquifaction) in saturated silty and sandy soil.

e. Erosion of the soil by seepage out of the bank at relatively low channel velocities.

f. Erosion of upper bank, river bottom or both due to wave action caused by wind or passing boats.

Deposition or bar formation occurs where the amount of sediment arriving at a location in the channel section exceeds the sediment transporting capacity at that point.

The complementary processes of bank erosion and bar formation result in meanders migrating downstream. The geometry of a particular channel may also be determined by natural hard points, that is, by geological formations which are highly erosion resistant.

A detailed discussion of meander migration and planform stability is beyond the scope of this report. A good introduction to the subject is given in Chang (1988). Meander migration is discussed in a later section as it relates to stable channel alignment.

2.3 Classification of rivers

When discussing river training works the planform is an important way of classifying rivers. An index used to describe channel planform is sinuosity. Sinuosity is defined as the ratio of the length of the valley to the distance along the riverbed.

In an early study of river planforms by Leopold and Wolman (1957) the three basic planforms distinguished were straight, meandering and braided. Later work by Brice (1983) and others has refined these rather simplistic definitions, but has retained the same framework. Figure 3 shows some typical river planforms.

A straight river is normally defined as one which has a sinuosity less than 1.5. Absolutely straight rivers do not occur naturally, but this is the preferred alignment of man-made canals.

A meandering river has a sinuosity greater than 1.5 and normally in the range 1.5 - 4.3. Meandering channels are formed of alternating bends of reverse curvature which can develop increasing amplitude and skewness eventually leading to neck cut-off and the formation of oxbow lakes. Figure 4 shows some of the features associated with natural meandering channels. The meander pattern tends to move downstream by a process of bank erosion and deposition. The historical progress of meander patterns often shows up on areal photographs.

Braided channels are formed of apparently randomly interconnected channels separated by bars and giving the overall appearance of a braid. The river as a whole can be visualised as the path followed by the outer limits of the braiding and usually has a straight alignment, whilst the interconnected channels are usually sinuous. It has been claimed that braiding often occurs in aggrading rivers when the sediment transported at high discharges is too great for continued transport during reduced flow. The material is deposited in bars which can become vegetated and stabilised thus increasing braiding during the next high discharge.

These classifications imply a characteristic discharge at which the planform is assessed since it is possible to find a river which could be described as meandering at very low flow, braided at higher discharge or almost straight during extreme floods, such as reaches of the Mekong River in N Thailand. The characteristic discharge is usually taken to be either the bankful discharge or the dominant discharge. But there is no commonly accepted definition of these terms and so an element of subjectivity inevitably creeps into planform classification.

The training of branched or braided rivers into a single channel is called channelisation. From a river authorities perspective, a river should not cause damage to property and should remain stable for use as a navigation channel. According to Petersen (1986), the underlying concept in river training is to shape a river into a single channel with a stable bed following a path of easy bends of reverse curvature, and to fix it permanently on that alignment. A range of river training structures are used to achieve this objective. Most major rivers of Europe have been channelised over the centuries. As an example, Figure 5 shows the training of the Waal in Holland from a sinuous braided form into single navigable channel.

2.4 Regime theory and dominant discharge

Regime theory refers to the process of self-adjustment of rivers in response to the physical laws of nature in order to achieve a state of dynamic stability. Blench (1969) describes regime theory as applied to rivers and canals. He defines "regime" as being: "... the behaviour of channel over a period based on conditions of water and sediment discharge, breadth, depth, slope, meander form and progress, bar movement, etc. ... it could be called the climate of a channel." The term 'in regime' means that the regime does not change over a significantly long period of time, i.e. that a state of dynamic equilibrium has been reached.

A river can adjust its slope, width, depth and meander pattern by aggradation or degradation of the bed and erosion or siltation of the banks. The regime conditions of a river with a bed of non cohesive material is dependent upon its sediment transporting characteristics. The rate of sediment transport varies with discharge. This has given rise to the idea of a 'dominant' or 'formative' discharge. This is a notional uniform discharge that would produce the same regime as a natural sequence of discharges.

Inglis suggested that "...there is a dominant discharge and its associated charge and gradient to which a channel returns annually. At this discharge equilibrium is most closely approached and the tendency to change is least. This condition may be regarded as the integrated effect of all varying conditions over a long period of time." Unfortunately, there is no universally agreed method of determining the dominant discharge. In a study carried out at HR a number of different definitions proposed for dominant discharge were compared using data from large gravel rivers in Canada. The data set was limited, but the results were best for those based on flow frequency. The discharge which is exceeded 0.6% of the time gave the best predictions (White et al., 1986).

The method for calculating dominant discharge quoted by Bognar and Hanko (1987) is based on the flood duration curve weighted by the corresponding sediment transport rate. The dominant discharge is then the discharge at which the most sediment is transported during the year. The equation is :

$$Q_d = \frac{\int QS\ dt}{\int S\ dt}$$

Regime theories can be categorised as being either empirical or analytical. Regime theory was originally developed in India as a way of designing stable irrigation canals where the water carries a significant amount of sediment. The most important early contribution was that of Lacey, who succeeded in relating all the geometric properties of a stable channel to its characteristic discharge and a "silt factor". Blench later extended these ideas to natural rivers and developed correlation equations covering a wide range of discharges and sediment sizes.

The functional form of the relationships has been determined from large data sets, such that the fit can look quite good when plotted on log-log paper. The applicability of any set of regime equations depends upon the similarity between the channel under investigation and those from which the equations were deduced.

The analytical approach uses a sediment transport formula and a resistance equation together with either a third equation such as bank stability or a variational principle. The assumption which has been used at HR is that a river tries to maximise its sediment transporting capacity. This approach is not truly analytical in that it uses sediment transport and resistance equations which are themselves empirical. To simplify the use of regime theory HR has produced a set of regime tables to predict equilibrium channel conditions (White et al., 1981). A table for each sediment size gives the equilibrium width, depth, velocity and slope corresponding to a range of discharge and sediment concentrations.

3. FUNCTIONS AND FORMS

3.1 Purpose of training

"River training works are improvement works undertaken by man to change or stabilise the natural tendency of a river in its depth, width and alignment in order to make the river serve man's purposes" (UNECAFE, 1953). Examples of the objectives of river training are :

(1) Improvement of flood flow passage.
(2) Improvement of navigation channel.
(3) Stabilisation of bank and river course.
(4) Stabilisation of river bed.
(5) Direction of flow through defined reach or past hydraulic structures.

River training works are sometimes grouped into three categories according to the flow conditions considered in the design objectives :

(1) High water training - also known as discharge training.
(2) Low water training - also known as depth training.
(3) Mean water training - also known as sediment training.

3.2 Types of river training works

This section lists all the main types of river training works together with a simplified description of their form, function, construction and design parameters. Different countries use different terms or sometimes different spelling for the same structures. This can be confusing when searching for information on any particular structure and so a list of equivalent names in other countries follows the usual UK terminology.

Figure 6 shows a typical layout of river training works for channelisation and Figures 7 to 13 show typical details of the more common structures.

<u>Impermeable groynes</u> (also known as Groins; Spurs; Spur dikes). See Figure 7.
Description - transverse structures extending from the bank into flow which completely block the flow over their length. They can be used singly or in groups.
Functions - to protect bank from erosion; to realign main channel along line traced by the tips of a group of groynes by promoting siltation between groynes; to deepen the main channel for navigation; to divert flow at a specific location eg an intake.
Construction - gabions; concrete blocks; rip-rap; sheet piles.
Design variables - length; spacing; orientation; crest elevation; scour protection.

<u>Permeable groynes</u> (also known as Jetties; Retards). See Figure 8.
Description - transverse structure projecting from the bank into the flow which allows some of the flow to pass through the structure.
Function - bank protection by reducing near bank velocities and promoting siltation.
Construction - timber piles; jacks; cribs.
Design variables - same as impermeable groynes; spacing of piles or jacks.

<u>Guide banks</u> (also known as Guide bunds). See Figure 9.
Description - artificial banks constructed upstream and downstream of a structure.

Function - to guide flow through a defined reach; to create favorable flow conditions at a structure; and to protect a structure from local scour.
Construction - gabions; rip-rap; rockfill.
Design variables - stone size; length; plan shape; extent of scour protection.

Cut-off walls (also known as Closures). See Figure 10.
Description - transverse structure constructed to completely block off a branch of the river.
Function - to close a channel branch in order to create a single channel for navigation, diversion or other reasons.
Construction - rockfill; prefabricated concrete blocks.
Design variables - rock or block size.

Longitudinal dikes (also known as Training walls; Bulkheads). See Figure 11.
Description - artificial banks constructed in front of and roughly parallel to existing banks.
Function - to re-align the channel or to protect existing banks.
Construction - gabions; sheet piles; rockfill; riprap.
Design variables - crest level; slope; stone size.

Flood embankments (also known as Bunds; Levees). See Figure 12.
Description - Raised embankments constructed parallel to the channel on the banks of the main channel.
Function - contain design flood discharge within a defined channel.
Construction - gabions; compacted earth; rockfill.
Design variables - crest level; freeboard; slope.

Bank revetment (also known as Bank protection; Facing material). See Figure 13.
Description - continuous covering placed over a natural bank.
Function - to prevent bank erosion by resisting erosive forces of river flow and wave action.
Construction - rip-rap; mattress; armouring; vegetation.
Design variables - stone size; length of toe protection; slope.

Bed protection.
Description - Protective layer placed over the bed of the channel.
Function - to resist scouring of bed material.
Construction - rip-rap; rockfill; bed panels.
Design variables - stone size.

Weirs and sills (also known collectively as Grade control structures).
Description - transverse structure with a fixed sill level across the full width of river.
Function - fix bed level and hence control longitudinal bed slope.
Construction - mass concrete; sheetpiles; rock.
Design variables - sill level; downstream scour protection.

4. IMPACT OF TRAINING WORKS

4.1 Environmental impact

River training works will change the natural conditions in the river and along the banks. Environmental considerations are becoming increasingly important in the design of all river works. The US Army Corps of Engineers (1981) listed the following environmental considerations when designing streambank erosion control projects and these are equally valid for other river training works :

* Aesthetics
* Water quality
* Biological impacts
* Physical impacts

The aesthetics of river training works are a major concern where a river is used for recreation or is part of an area of natural beauty. A recent development is a return to using natural materials, such as timber, for constructing training works, because these blend well into the visual environment. Fascines, for example, are bundles of sticks or twigs, often of willow, tied together to form mats. The use of traditional methods of creating a stable channel using fascines is described by Vischer (1987), and also by UNECAFE (1953), and the use of vegetation for bank stabilisation is described in the report by the US Army Corps of Engineers (1981).

An important impact of groyne fields is capturing silt between them, but along with the silt can come pollutants such as heavy metals. Westrich (1988) studied the impact of different groyne designs on pollutant transport along the Rhine. He found that the groyne fields acted as concentrators of pollutants at low flows and then would release them at high discharges as the flood flow washed-out the pollutant bearing silts.

Shields (1983) summarised the available information regarding the environmental aspects of longitudinal dike field design. The trend in the USA of building continuous stone embankments has changed miles of riverbank environment. As long as dike fields remain aquatic they provide valuable habitat for fish and macro-invertibrates. The crux of the environmental design problem is to stop pools behind the dikes silting up completely. The most common way of achieving this objective is notching. He recommends that a range of notch sizes and configurations be used in order to create a diversity of habitats.

River training works are intended to have a physical impact at a specific location. The problem is that there may be unforeseen local effects in addition to those intended and the training works may have an impact at a much larger scale due to backwater effects and changed sediment transporting characteristics.

4.2 Local physical impacts

The most important local impact of training works is scour. Scour at a river cross-section can be thought of as being made up of general and local scour. River training works which increase the average flow velocity at a section will tend to increase general scour. Increased average velocity can be caused by either a channel constriction or decreased flow resistance at a section. In addition to increasing average velocities due to constriction, river training works which obstruct the flow give rise to large scale eddies and small scale vortices. This in turn can lead to scour or deposition depending upon the relative change in velocities. Increased curvature of flow at the bank will lead to increased bank erosion. Depending upon the orientation of an obstruction to the flow, an area of relatively sluggish flow is created either upstream or downstream between the separation streamline and the bank. In general, reduced velocities in this region lead to deposition, but eddies can form, which may increase local bank erosion.

Complex vortex systems are created around groynes and guide banks. The pattern of vortices around groynes observed by Copeland (1984) is shown in Figure 14. Copeland noted that the small scale vortex system increase bed shear velocity leading to local scour. A strong primary vortex is formed at the nose of groynes and guide banks, causing the deepest local scour hole at that point.

The local physical effect of each type of training works can be summarised as follows:

Groynes - strong local impact.

- Local scour at nose due to primary vortex.
- Local scour at upstream and downstream faces due to intermittent weak vortices.
- Sedimentation in low flow zones behind, in front or between groynes.
- General scour due to channel constriction.
- Diversion of streamline may promote erosion of the bank opposite or adjacent to groynes.

Guide banks - strong local impact.

- Constriction increases general bed scour.
- Local scour at upstream nose.

Cut-off - strong local impact.

- Width of main channel will tend to widen by closure width.
- Diversion of streamlines will cause attack on bank adjacent to closure.
- Dead water behind closure may silt up.

Longitudinal dikes - strong or weak local impact depending on degree of constriction.

- Erosion at toe of dike bank.

Flood embankments - weak local impact.

- Increased water level at flood discharge.
- Increased bed degradation at flood discharge.

Bank revetment - weak local impact.

- May increase or decrease boundary roughness depending upon materials.
- Increased erosion at toe of bank.

4.3 Large-scale physical impacts

Attempting to fix the width, depth or alignment at a given location may affect river morphology and flood levels both upstream due to backwater effects and downstream due to changed sediment transporting characteristics at the upstream boundary. Any works at a channel section can change: (i) stage-discharge relationship; (ii) sediment transporting characteristics; and (iii) velocity distributions across the section.

The dominant channel forming variables relate to discharge and sediment transport. The US Army Corps of Engineers (1981) stated that because of the interdependence and variability of factors every river has to be treated as a unique and special case. Nonetheless, they listed the general response of rivers to changes in boundary conditions in terms of aggradation and degradation. These are given below.

		Common response of channel	
Factor	Change	Action	Response
Peak flow rate	Increase	Degradation	Strong
(quantity)	Decrease	Aggradation	Weak
Flow duration	Increase	Degradation	Strong
(high flows)	Decrease	Aggradation	Weak
Sediment yield	Increase	Aggradation	Weak
(basin)	Decrease	Degradation	Strong
Sediment transport	Increase	Degradation	Strong
capacity (river)	Decrease	Aggradation	Weak

At present there is no equivalent prediction of general planform response to change.

Meander migration rates are usually very slow, typically a few centimetres a year, and so planform response to change may take years to become apparent. In contrast, significant bed level changes can occur during a single flood. Examples of planform responses to engineering works are shown in Figures 15 to 18. Figure 15 shows the response of reaches of the Mississippi to different arrangements of groynes and bank revetment.

Any training works which constrain the migration of meanders at a particular location will generally cause the river to "concertina-up" upon the constraint. This is illustrated by Figures 16 and 17c. But Figure 18 is an example of entirely different planform response to a constraint on the same river. Grissinger and Murphy (1983) used this example to show the importance of local lithology in determining planform response. Despite the difficulties, for planning purposes we want to be able to predict the physical response of the river to building training works.

5. PREDICTING PHYSICAL IMPACTS

5.1 Simple formulae

Several authors have proposed using regime theory to predict channel response to training works. Blench (1969) discusses the use of his regime equations to predict the impact on dependent channel variables of building cut-offs, guide banks and groynes. His four basic regime equations relate the four dependent variables - width, depth, slope and meander length, to four independent variables - F_b, F_s, Q and k. A change in one dependent variable, for instance by constricting the width to a reduced value b, will increase the depth according to the appropriate regime equation. Blench's equations relate the meander length M_L only to the regime channel width B:

$$M_L = 10B$$

This equation was derived from field measurements from a large number of rivers and for a range of discharges. It follows that the effect of channelising a river and reducing its width should be to increase its meander length.

Bettess and White (1983) used regime equations to predict river planform as measured by sinuosity and the impact of change on planform. They assumed that planform results from a compromise between valley slope and the equilibrium bed slope predicted by regime theory.

Blench also describes general rules for estimating local scour. A practical way to calculate scour, based upon Indian experience, it to visualise any obstruction - abutment, groynes, etc - as causing a fractional enhancement of ordinary straight channel regime depth. Based on the work of Inglis, from a study of extreme scour around bridge piers, he proposed that the maximum scour depth is twice the regime depth estimated for high floods.

There is a philosophical difficulty in using regime equations to predict local response to training works. The problem is that these equations were derived for dynamic equilibrium conditions in long reaches of natural channels. Training works disturb that equilibrium and the regime equations are, strictly speaking, no longer valid.

Several researchers have proposed general and local scour formulae based upon theoretical, experimental or field studies. These formulae are summarised in Appendix B. The problem is that they are only valid for the particular geometry and sediments for which they were derived.

5.2 Physical models

The impact of training works on plan shape can be studied using mobile bed models. Physical models remain the most common way of studying the local impact of training works. The problem with mobile-bed physical models is that of scaling. The scaling laws for Froude-similitude between model and prototype are explained by Novak and Cabelka (1981). Dhillon et al (1980) discuss the scaling laws for steady state mobile bed models and describe two model studies of river training works carried out in India.

The problem with mobile bed models is that it is difficult to simulate both hydraulic roughness and sediment movement simultaneously. The approach followed by Dhillon et al for modelling alluvial rivers is based on Lacey's regime relationships. They chose the length scale based purely on practical consideration of space, cost and discharge. The vertical depth scale then follows from Lacey's formula:

$$D_r = F_1^{0.33} L_r^{0.67}$$

where F_1 is Lacey's silt factor. They state that a further check should be made using the Manning's law relationship:

$$D_r = L_r^{0.75}$$

but they do not indicate what to do if the two scales are in significant disagreement.

The mean sediment size is determined from a critical tractive force at the dominant discharge for which the model will be run:

$$d_{50} = 0.01 \quad t_c$$

They follow the advice of Blench and Lacey by using bed material of the same density as the prototype. Other approaches to scaling bed load movements include the use of large size, lightweight materials, scaled in accordance with Shield's law for threshold of movement and one of the sediment transport formulae.

One case study described by Dhillon was a scale model of an embayment and unfavorable flow conditions at the guide banks for a railway crossing. The recommended remedial measures included constructing cut-off walls to the existing meander bends and an artificial pitch island to direct flow more evenly through the three spans of the bridge. They indicate that the prototype did not perform as well as the model and suggested the following reasons:

1. Lack of reliable field data.
2. Non simulation of grain size distribution.
3. Different timescale for morphological scales between model and prototype.
4. Uncertainty in selecting a dominant discharge for running the model.

Problems 2 and 3 are common to all physical models.

A series of physical model studies of channel stabilisation of the Rio Mamoré river in Bolivia carried out al Leuven and Torino were described by Berlamont and Schiara (1983). They studied the problem of stabilising a single meander bend along a meandering "reach type" river in order to build a port on a concave bend. By 'reach type' is meant that the meander amplitude and wave length remain constant whilst the whole meander pattern moves downstream.

They found that stabilising only the bank on which the port was constructed meant that as the meander pattern migrated, the port was eventually left high and dry. Their progress towards a solution is shown in Figure 17. Stabilising the concave bank of the meander bank upstream was also ineffective. Protecting both the concave and the convex bank opposite the port with groynes, maintained the deep water channel for navigation, but the meander loop upstream folded up upon the groynes threatening to outflank them. The solution they arrived at was to construct a funnel shaped combination of training work into which the meander wave disappears without forming sharp bends. But construction of bank protection measures could be spread in stages over a number of years to reflect the timescale of morphological change.

An interesting aspect of this work was that the models at Leuven and Torino were constructed at different scales, most notably the bed material size. One model was considered as "a model" of the other and used to test the scaling laws for the morphological timescale, as measured by the meander migration rate, and other prototype parameters predicted from Froude-similarity equations. Agreement between model and prototype was remarkably good.

There are many examples of the use of mobile bed models to study local scour around structures. The maximum depth of scour is obtained if the model test is conducted at the threshold of movement for the bed material used. A difficulty reported by Copeland (1983) and by other researchers is that an armour layer develops in the scour hole which effectively limits the depth of scour. He found that the armour material in the scour hole was <u>all</u> greater than d_{95} of the medium sand used for his experiments and much of the material was larger than the maximum sediment size determined from the original grading curve. This shows the importance of considering the complete range of sediment sizes rather than just a single "representative" grain size when studying local scour.

There are serious experimental difficulties in reconciling the various scaling laws for both flow similarity and similarity of bed load movements. There is also the problem that backwater and associated morphological effects can extend for several kilometers upstream of training works at, say, a bridge crossing, but economic and practical constraints usually dictate that models cannot be too large.

Physical models are still the best way of qualitatively studying the local impact of river training works because they reproduce the physics correctly, but problems can occur due to scale effects both from secondary currents and sediment scale distortions. It may not be possible to run the model for a realistic range of discharges over a sufficient period of time. Numerical models offer the advantage of flexibility in that they can be run for actual discharges and a range of configurations to compare options.

5.3 Computer models

Mathematical models have played an increasing role in hydraulic research since computers started to become widely available in the sixties. The different types of morphological river models can be characterised by the number of spatial dimensions in the flow model. A summary of the different morphological models reported in the literature is shown in Table 1. A morphological model links a sediment transport and continuity model to a flow model.

The only proven morphological model widely available to-date is the one-dimensional (1D) model. The common assumptions behind 1D morphological models is that the flow is essentially 1D. Quasi steady-state 1D models are the simplest form of morphological models. Backwater calculations are used to calculate steady states, mean velocity and depth along the river. Bed level changes are then calculated from the difference in sediment transporting capacity between adjacent sections using a sediment transport relationship such as Ackers-White and the sediment continuity equation proposed by de Vries. The backwater calculations are then repeated for the updated bed levels and the whole process continues. Bed aggradation or degradation is approximated by a series of steady states.

These models have been used to study reservoir sedimentation problems and the problem of bed degradation downstream of a dam. One dimensional models have reached a high degree of sophistication and can treat the effect of variable sized bed material and flood duration curves to study long term change. For a description of the MORMODEL model developed at Hydraulics Research see Bettess and White (1981). This model has been applied to a wide range of applications. Jansen et al. (1979) gives examples of the use of 1D morphological models developed at Delft Hydraulics to study aggradation and degradation at contractions, cut-offs and general scour at a bridge constriction. Figures 19 to 21 show examples of model results.

An unsteady state 1D model MOBED developed at NWRS, Canada, is described by Krishnappan (1985), for predicting bed level changes in response to artificial meander cut-offs. The model uses Preissemann's scheme to solve the unsteady flow equation and uses the Ackers-White sediment transport formulae. The programme TABS-1, written by the US Army Corps of Engineers, also has all these features.

There are several forms of two dimensional morphological models. Depth average 2D models solve the St. Venant equation to give the two-dimensional plan flow vector at a number of points across the width as well as along the length of a channel.

Conceptually the model then proceeds as for a 1D model except that a careful track must be kept of where sediment is coming from and going to in order to ensure mass conservation.

Two-dimensional flow at river bends and associated bed profiles and sediment grading are reported as having been successfully modelled using the Delft Hydraulics Programme RIVCOM, the programme FLUVIAL, described by Chang (1988) and by the US Army Corps of Engineers' model TABS-2, described by Thomas and Heath (1983). The TABS-2 model uses a finite element formulation and the necessary geometric details can be achieved even at the level of a single groyne. Figure 22 shows a sample output of velocity vectors from TABS-2 applied to training works on the Mississippi river.

Another form of 2D model is the river meander model used by Parker (1983) to study bend stability. This combines the meander parameters of radius of curvature and meander length with the width, depth and mean velocity. He used a simple erosion-deposition model, based on bank erodability and velocity variation across the section, calculated as a function of distance from the channel centre line. His analysis resulted in a convolutional relationship between migration rates and channel curvature. The form of the relationship was such that tight bends subside, and bends of long wave length grow in amplitude.

A completely 3D model is only practical for small scale flow modelling around structures. A 3D model with turbulence representation would be required to model detailed erosion patterns around, for example, a submerged groyne. Fully 3D flow models are commercially available. e.g. PHEONICS as described by

Moscardini (1984), and are widely used in the chemical process industry. Open channel 3D morphological models are under development by the US Army Corps of Engineers, TABS-3 reported by Thomas and Heath (1983), and Delft Hydraulics, but as yet there are no published examples of their successful application to river training works.

6. REVIEW OF DESIGN INFORMATION

6.1 The design procedure

Any plans for training works need to be evaluated both in terms of achieving local objectives safely and economically, and in terms of larger scale impacts. The design procedure involves first defining the problem and objectives to be met by constructing training works, then evaluating different options for achieving the objectives both locally and in terms of large scale impacts. Finally, comes the detailed design of the selected works. Figure 23 shows a flowchart of the design procedure.

Locally training works must be strong enough to resist the design velocities and must extend beyond the area of potential scour to safeguard against undermining. The detailed hydraulic design of training structures to resist a design flood is outlined in Bognar and Hanko (1987), and in the manual by Brandon (1989). A major practical difficulty is in determining design flow velocities at the bed and banks for which structures must be designed.

Design options at the planning stage and ways of assessing probable channel response are covered by Chang (1988).

In many cases the key design problem is that of assessing planform stability of existing and proposed channels.

6.2 Stable channel alignment

River training structures can be used to realign or channelise the river. This may mean selecting a new section size and planform. If a revised planform is unstable this will lead to problems and high maintenance expenditure. If the selected curvature is too low, then sand bars may form so as to increase the sinuosity. If the channel is to be used for navigation this may result in high dredging requirements. If the curvature is too high, then the flow may attack the bend and eventually destroy or outflank the training works. Examples of channelisation are shown in Figure 24. The use of bends of insufficient curvature in Figures 24a and 24b for the Rhine led to propagating sand bars as shown on Figure 24c.

The variables to be selected when designing stable planforms are: width; depth; sinuosity; bend radius; curve arc; and transition length. Rules based upon practical experience from channelising European rivers are given by Bognar and Hanko (1987) and also by Jansen et al (1979). Their recommendations are summarised in Figures 25a and 25b and Table 2, along with the recommendations of other authors. The recommendations are mutually contradictory and it is impossible to select the best advice without further information.

An analytical treatment of channel stability was given by Parker (1983). One of his conclusions was that all natural bends migrate downstream. This has been found by several researchers in laboratory flume experiments and is a phenomena well known to geographers who look at river processes at a longer timescale than engineers. Field studies of meander parameters and migration rates for several rivers in the US were carried out Nanson and Hickin (1983). If it is true that all bends in alluvial rivers tend to migrate downstream then in the long term some form of bank stabilisation will be necessary along the entire length of a river in order to fix it permanently on a particular alignment. This has been the lesson of several hundred years of river training on the major rivers of Europe. Whether this form of channelisation is desirable and what alternative more environmentally friendly river management options are discussed by Brookes (1988).

6.3 Groynes

The design should follow from the function the groyne is to serve. Various types of groynes are illustrated in Figures 7, 8 and 26. Groynes can be used to :

* Protect a bank against erosion;
* Channelise or guide flow into a preferred alignment;
* Establish a deep navigation channel.

The choices to be made when planning and designing groynes are:

1. Single groyne or groyne field;
2. Permeable or impermeable groynes;
3. Length of groynes;
4. Angle of groynes to bank;
5. Spacing of groynes;
6. Plan shape of groynes;
7. Crest level and crest slope of groynes;
8. Extent and placement of scour protection.

This literature survey has shown that none of these questions can be answered definitively, but research results and practical experience from several countries do offer some guidance.

Alvarez (1989), Bognar and Hanko (1987) and Salikov (1987) give quite clear recommendations on each design parameter based upon research and practical experience in Mexico, Hungary and Russia respectively. The problem is that these recommendations are sometimes contradictory and clearly derived for specific conditions, such as Salikov's advice that the optimum groyne spacing is 50m. Thus there is a danger of generalising from particular conditions, and without access to the data on which these recommendations are based it is impossible to be sure of their range of validity.

Single groynes and groyne fields.

A single groyne (called a "long spur" in India) used without any other bed or bank protection can be dangerous since the diverted flow can attack the bank and cause increased meander development. In general groynes should not be used in isolation. Nonetheless, isolated groynes have been used to protect canal intakes, guide flow at bridges and protect a length of river bank. There have been some dramatic failures of large single groynes. Garg (1980) quotes the example of the Kankahl long spur on the River Ganges in India which was constructed in 1951 pointing downstream to deflect current into the main channel. It failed within a few years due to erosion of the upstream face and outflanking. A new groyne was constructed pointing upstream.

A theoretical study of an isolated groyne was made by Tödten in 1975 as reported by Bognar and Hanko (1987).

His analysis was based upon the laws of streamflow in a plane. He considered a perpendicular orientated groyne of length b projecting into a long straight channel. A definition sketch is shown in Figure 27. A separation streamline exists starting at the tip of the groyne and reattaching itself to the bank a distance L downstream. The distance S_L can be thought of as the length of protection afforded to the shore by the groyne, although in fact an eddy may form in the recirculation zone and cause bank erosion. There is little information available upon the strength of this recirculation zone or methods of predicting whether bank erosion will take place. The separation streamline is the boundary between the mainstream and the recirculation zone. He found that the maximum width of the recirculation zone b^* is $1.67L_t$ and the maximum length of S_L is $12.5L_t$. As the shape and length of the recirculation zone must depend upon the flow conditions it is clear that such a relationshop can only have a limited range of applicability.

Nwachukwu and Rajaratnam (1980) conducted two series of experiments on single perpendicular groynes with normal approach flow. The first set of experiments looked at the distortion of the flow field and corresponding bed shear stresses for two lengths of groyne and a variety of approach depths and velocities. These experiments were made in a fixed rough bed flume using an instrument called a yaw-probe to map the 3-dimensional velocity and shear stress fields. The second set of experiments used a mobile sand bed flume to study local scour patterns around similar groynes. The report contains a lot of data about extent and development of the scour hole, but many of the results are not in a useful form.

Referring to Figure 27 they found that for a thin plate groyne the separation streamline was very similar to that predicted by Tödten and could be satisfactorily analysed by the model of a skewed turbulent boundary layer. The outer boundary plane of the shear layer is not vertical, but tilted towards the recirculation zone near the bed. An important consequence of this is that a depth averaged 2D flow model with an assumed logarithmic velocity profile could not represent this flow field correctly. The dimensions of the recirculation zone for the thin plate groyne were : $b^* = 2L_t$ and $S_L = 12L_t$. But S_L reduced to $5L_t$ when a thicker cylindrical groyne was used. The effect of the flow disturbance extended about $2L_t$ upstream.

The maximum bed shear stress occurred at the nose of the groyne. For the range of constrictions studied they found that the maximum bed shear stress compared to the bed shear for undisturbed flow increased almost linearly with blockage ratio. Based only upon two blockage ratios their data gives a shear stress amplification of:

$$t_{om}/t_{oo} = 1 + 23.5L_t/B \text{ for } 0 < L_t/B < 0.17$$

The extent of bed shear stress amplification compared to the maximum value at the groyne nose was contoured for each experiment. Figure 28 shows sample results. The shear stress amplification extends across the section at the groyne because of the constriction, but superimposed upon this is a local maximum at the groyne nose caused by the primary vortex. The local distortion extends about $4L_t$ downstream and $2L_t$ across the channel from the groyne nose. It seems likely that in general these dimensions vary with the flow conditions.

Mobile bed scour tests were carried out for clear water scour only. The maximum scour depth occurred at the nose of the groyne and was found to depend mainly on the constriction ratio. The scour hole progressed upstream from the location of maximum bed shear at the groyne nose by a process of undermining and slumping.

They measured the extent and development in time of the scour hole and found that the history of scour showed three distinct phases: an initial rapid scouring; a linear-log phase; and an end process. Their results are shown in Figure 29. The geometry of the scour hole was consistent for all phases. They did not produce predictive formulae for either the extent or time of scour.

Groynes should be used in groups or "fields" to fix the limit of flow of a river along the line defined by the tip of the groynes. The upstream groyne should be aligned so as to cause minimum deflection of the flow, often being combined with a longitudinal dike to give a smooth transition at the groyne field. A typical arrangement of groynes for channelisation is shown in Figure 6. All authors agree that the first groyne should be placed at the start of the bend for bank protection works.

Permeable and impermeable groynes

Permeable groynes are used as a bank protection measure. Their advantage is that they are relatively cheap and cause less disturbance to flow patterns than impermeable ones. But they are less robust and can be destroyed by floating debris or the drag forces due to trapped debris. Permeable groynes are used to encourage siltation by reducing flow velocities within the groyne field without causing a major diversion of the streamlines. There are many different types of permeable groyne ranging from timber piles to wire 'jacks'. Typical construction details of the various forms of permeable groyne can be found in the US Army Corps of Engineers manual on bank protection (1981).

Miller et al (1983) carried out a series of 53 model tests on permeable and impermeable isolated groynes to study the effect of orientation, length and crest elevation on scour, length of bank protected and velocity reduction. Their conclusions were mainly qualitative and confirmed what would be expected : permeable groynes reduce shear velocities compared to impermeable groynes, and scour depth decreased as the groynes were inclined more downstream. It perhaps should be observed that a designer requires quantitative information.

Length of groynes

According to the results of Nwachukwu and Rajaratnam (1980), the length of a single groyne needed to protect a length of bank S_L downstream is between $L_t/12$ and $L_t/5$. The results of the work of Miller et al (1983) indicated that the ratio of the length of bank protected to the projected length of the groyne was a maximum for a constriction ratio b/B of 0.78. It is likely that the length of bank protected also depends on the flow conditions. However it must be repeated that most authors recommend against the use of single groynes.

For groyne fields, the tip of the groynes define the margin of the new channel alignment. Alvarez (1989) recommends that the length of groyne in the flow should lie within the limits : $d \leq L_t \leq 0.25B$. Salikov (1987) recommended a projected groyne length not exceeding 15-20% of the width, ie $b/B \geq 0.8$. The length of the groyne is then the length of groyne in the flow L_t plus an embedded length L_e. The embedded length is to allow for bank erosion at the junction of the bank and the groyne. Alvarez recommends an embedded length L_e of $0.25L_t$. Copeland found that for the perpendicular groynes in his model tests the total scoured length of the groyne from its tip to the bank after testing depended upon groyne spacing. Figure 30a shows that the ratio $L_t/(L_t+L_e)=0.7$ upto $S/L_t=4$ and thereafter $S/(L_t+L_e)$ is constant. This supports Alvarez's recommendations.

Angle of groynes to bank

The orientation of a groyne relative to the bank is defined by the angle i between the downstream bank and the groyne as shown on Figure 26. The choice of this angle and whether groynes should be orientated pointing upstream or downstream has been a question of controversy for many years. Upstream pointing groynes are sometimes called 'attracting' and downstream pointing groynes are sometimes called 'repelling', referring to their deflection of flow along the upstream bank.
Sethi (1960) refers to "repelling" and "attracting" groynes in terms of their isolated effect in a straight channel. A downstream pointing groyne

deflects flow towards midstream and tends to cause erosion of the upstream bank adjacent to the groyne. A groyne orientated upstream creates an area of dead water or a slowly circulating eddy just upstream which tends to cause siltation. Strom (1941) argued that groynes orientated downstream should only be used in groups so that the downstream protection afforded by each groyne extends to the one downstream.

A summary of some of the recommendations on groyne orientation is given in Table 3. Recommendations are contradictory and confusing. The reason for adopting an angle other than 90° is a belief that this significantly affects the extent and depth of scour. In a comprehensive review of river training practice at the time, UNECAFE (1953) reported that groynes should be orientated either perpendicular to the bank or pointing upstream at an angle i between 100° to 120°. The orientation of a groyne affects both the location of the scour hole which forms towards the tip of the groyne and the maximum depth of scour. Kinori and Mevorach (1984) reporting on work by Gill present a graph which suggests that the scour depth relative to a perpendicular groyne increases for upstream orientation and decreases for downstream orientation. This is supported by the findings of Copeland (1983) whose results are shown in Figure 30b. But the figure shows that scour depth is relatively insensitive to groyne angle in the range 75-105°.
All authors agree that it is projected length perpendicular to the flow which is the important parameter in providing bank protection. After a thorough literature review and several experimental tests Copeland concluded that there was no good reason not to choose the most economic solution - groynes perpendicular to the flow.

Spacing of groynes

The spacing of groynes should be the maximum possible which will provide adequate bank protection. A summary of recommendations made by various authors is given in Table 4. For economic reasons the maximum possible groyne spacing should be chosen.

Most authors recommend spacings based upon the idea of a protected length of bank, ie how far apart should groynes be placed such that the protection afforded by each groyne extends to the one downstream. Thus Copeland used the eroded length of groyne after testing as the criterion for recommending spacing to length ratios of up to 3.

A different approach is taken in Holland, where optimum groyne spacing is seen in terms of energy dissipation. Groyne spacing is chosen so that one strong eddy forms between the groynes as shown in Figure 31a. From experimental work at Delft Hydraulics Laboratory, reported by Jansen et al (1979), they found that the spacing could be expressed as an energy equation :

$$S = C_d d^{1.33}/2gn^2$$

where the empirical energy loss coefficient C_d was found to be about 0.6. This equation can lead to very large spacings and in practice Kinori and Mevorach recommended using this value as an upper bound.

Plan shape of groynes

There are a wide range of plan shapes for groynes reported in literature, such as straight, T-head, bayonet and L-shaped. Figure 26 shows some of the possible forms. There does not appear to have been any systematic study of the relative performance of different groyne plan shapes.

Garg (1980) discusses some of the different forms which have been used in India and describes how many T-head groynes have had to be modified after construction due to heavy scour. His conclusion was that there is no significant advantage in using specially shaped groynes over straight groynes. Franco (1967), when testing model groynes with L-heads, found that the addition of L-heads tended to lower the performance of the groynes according to his performance rating system. His performance rating was based upon a combination of five factors: a dredging index; a scour depth; channel alignment; deposition between groynes; and main channel depth.

Crest level and crest shape

Groynes are sometimes constructed with their crests sloping downwards away form the bank and with relative crest levels of adjacent groynes either stepped upwards or downwards in the direction of flow.

Groynes can be designed to have their crest levels above or below the design water level. Most authors recommend that crests are above mean water level so that they do not become a hazard to navigation or recreational use of the river.

Alvarez (1989) recommends that groyne crests should slope downwards from the bank to the riverbed. He recommends crest slopes of between 0.1 to 0.25 and lists the following advantages of this design:

(a) Practically no local scour at groyne nose.
(b) Less material to construct compared to horizontal crest.
(c) Quicker sand deposition between groynes.
(d) No problems of outflanking provided spacing is less than four times the length.

Franco (1967) carried out a series of tests on groynes for the Mississippi River with different crest slopes and either horizontal, stepped-up or stepped-down crests in the direction of flow. Model groynes were constructed in an artificial stream at a scale of about 1:60 compared to the Mississippi. He compared performance of different systems in terms of his own performance index mentioned in the previous section. He concluded that sloping crest groynes can be designed to be as effective as horizontal crest groynes and that groyne systems which are stepped down relative to one another in the direction of flow are more effective than horizontal crest groyne systems.

<u>Extent and placement of scour protection</u>

The protrusion of a groyne into the streamflow will suddenly divert the flow and simultaneously constrict the channel. This sets up a complicated flow pattern around the structure. There will be a head difference between the upstream and downstream faces of the structure which will cause a local velocity increase at the nose and increased turbulence intensity. Complicated eddy systems will be set up around and between structures. The net result of these flow effects is an amplification of bed shear stress around the groyne compared to the undisturbed flow. If the shear stress is above the threshold value for movement of the river bed material, then scour will occur.

Scour can be thought of as being composed of general scour due to an increase in mean section velocity at the constricted section and local scour due to local variations in the velocity field such as in the vortex at the nose of a groyne. General and local scour formulae to predict the depth of scour at a groyne have been developed by several researchers and a selection is given in Appendix A.

The design problem is to estimate the area over which scour protection is required and the way in which scouring forces can be resisted. The usual way of achieving this is to place large rocks of a size sufficient to resist the amplified bed shear stresses. The commonly used formula quoted by Bognar and Hanko for rock size to resist a given maximum local velocity V is

$$D = AV^2$$

where D is the rock diameter and A is an empirical constant. Izbash (1935) recommends a value of A = 25. Neill (1973) has produced a graph from which the required rock size to resist a given velocity can be read off directly. The problem is then to estimate maximum velocity. Bognar and Hanko give a complicated procedure for calculating the increase in velocity due to both the sudden change in direction of the flow and the increased turbulence. A simpler approach is proposed by Salikov who recommends that the maximum design velocity can be calculated from:

$$V_{max} = 1.25V_{cv}$$

Where V_{cv} is the depth averaged velocity at the foot of the concave bend in the absence of the groyne. In reality it is likely that the increase is a function of the blockage due to the groyne.

6.4 Guide banks

The guide bank system was originally developed in India as a way of protecting bridge abutments from severe scour and to prevent extreme meander bends forming and attacking the approach bank to the bridge. According to Garg (1980), the guide bank system was developed by J R Bell in 1888. Later improvements were made by Spring, Gales and Sharma. He explains the background to the development of the guide bank system in India after the failure of other training works such as groynes to protect bridge piers.

Sethi (1960) summarises the Indian state-of-the-art at that time, which was based upon the work of Lacey, Inglis, Spring and Gales who used a combination of regime theory and practical experience. These recommendations were very specific concerning dimensions, but are strictly speaking only valid for the regime conditions in which they were developed. A typical guide bank layout is shown in Figure 9. The received wisdom was that guide banks should be straight and parallel along their central section with circular arc noses.

Neill (1973) brought together design information from a number of authors on design parameters for guide banks, including width between banks; length; plan shape; cross section; and height. His recommendations were based largely on work from Russia and again favoured straight guide banks through the bridge opening.

Garg (1980) describes a series of scale model tests to study the effect of the planform of guide banks on the worst meander loop which may form upstream and threaten bridge approach embankments. His conclusion was that "best" results were achieved by having the guide banks elliptical in plan shape rather than straight and parallel. His results are shown in Figure 32 and his recommendations on plan shape are shown in Figure 33. Alvarez (1989) regards guide banks as a special case of isolated groynes. He follows Garg in recommending an elliptical plan shape.

6.5 Cut-off walls

Cut-off walls are constructed across a branch or even the main channel to close off flow in that reach. Once constructed the cut-off wall becomes an artificial bank to the river, albeit with a very direct angle of attack by the flow. Once in place the design of a cut-off wall is identical to a rockfill longitudinal dike, ie it is simply a question of estimating the stone size to resist the shear force of the flow. But unless the cut-off wall can be constructed at very low flows, the critical design condition comes during construction when critical flow starts to form over the crest.

A full treatment of this topic based upon theoretical analysis, research tests and field projects on large Russian rivers is given in the textbook by Izbash and Khaldre (1970). This book explains a method for sizing rockfill or prefabricated concrete elements to resist the erosive action of a variety of flow conditions : flow past a rockfill contraction; flow over a rockfill weir; and turbulent seepage through a rockfill dam. The stability criteria are based entirely on stream power concepts and the critical mean velocity to just move a given stone size. Many of the equations have been empirically determined and verified and are relevant to the design of other river training works using rockfill. Cutting off a river channel and diverting all the flow down one or more other channels may have a significant impact on the overall channel pattern.

7. CONCLUSIONS AND RECOMMENDATIONS

All literature reviews have limitations, most notably time and access to material. This review has concentrated on published papers and reports in English. However, the papers by Alvarez, Bognar and Hanko, Salikov and Pilarczyk et al, give some information on design methods in their respective countries. The picture that emerges is that current design practice relies heavily on national experience and local prejudice. This is evidenced by the fact that the required groyne spacings to protect a length of concave bank vary by a factor of 5, between that recommended by Bognar and Hanko and by Salikov, although each are supposedly based on exhaustive tests.

The conclusion must be that existing recommendations are only valid for the particular conditions - field or laboratory tests - for which they were derived. What can computer models offer? Numerical modelling techniques have developed at a rapid pace over the last 10 years. Programmes such as **Pheonics** can solve the three dimensional Navier-Stokes equations including k-n turbulence modelling. Several groups are pushing ahead to develop 3D morphological river models which will rumble away on supercomputers to produce very impressive 3D plots of bed-levels and flow patterns. But do we yet really understand the processes involved? The structure of velocity, turbulence and sediment transport in real river bends is still poorly understood. Even the best sediment transport formulae claim an accuracy of only "half-to-twice" under field conditions. No-one has yet produced a practical theory of riverbank resistance, erosion and stability which can take account of vegetation and relative compaction.

Nonetheless, work is underway at Delft and other hydraulic Institutes to combine the best representations of these processes presently available into computer models for predicting river response to training works. Local models of flow around structures must be fully 3 dimensional and capable of reproducing the small scale eddies and velocity profile inversions observed in reality and they must use sediment transport equations which take account of local variations in sediment concentration profiles rather than a total load formulation. Global models of meander migration need to look at a much longer length of channel. A one-dimensional model is adequate for calculating average discharge and water levels along several kilometers of river, but for erosion and deposition processes the curved nature of the flow needs to be considered.

In order to improve our ability to predict the impact and performance of river training works, specific recommendations for strategic research at HR are:

1. Conduct basic research into river bank erosion and meander migration. River banks differ from mobile beds in being more cohesive, compacted and usually vegetated. A field study should try to establish empirical relationships between descriptive indicators, such as soil type, angle of repose and degree of vegetation, and an erodibility index. Actual erosion rates would then need to be related to the erodibility index and flow and bend parameters.

2. Extend the existing HR 1D morphological computer model to include radius of curvature and bank erodibility - a sort of 1½D model - along the lines of that described by Parker (1983).

3. Develop a 3D morphological model of local flow and erosion around structures. A commercially available 3D model which includes turbulence and a free surface, such as Pheonics, could be used to solve the flow field rather than developing this from scratch. Work at HR would concentrate on including appropriate forms of the sediment transport and continuity equations.

Both computer models should be tested and verified against first laboratory and then field data.

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TABLES.

Table 1 Computer based morphological

Name	Dimension	Flow model	Sediment transport	Organisation	Reference	Comment/applications
MORPH	1 d	steady	Ackers-White	HR, UK		Degradation downstream of dam
TABS-1	1 d	steady	Ackers-White	USACE, USA	Thomas & Heath (1983)	
MOBED	1 d	unsteady	Ackers-White	NWRI, Canada	Krishnappen (1985)	Artificial cut-offs
RIVMOR	1 d	steady	van Rijn	DHL, Holland	Delft Hydraulics (1987)	Bed level change due to wide range of river works
MIKE-11	1 d	unsteady	Several options	DHI, Denmark		Commercial package
FLUVIAL	1-2 d	unsteady	depends upon conditions	San Diego Univ USA	Chang (1988)	Design of training works
MORPH-2D	2 d depth av	steady	Ackers-White	HR, UK	Penn (1989)	Reservoir sedimentation
RIVCOM	2 d depth av	steady	van Rijn	DHL, Holland	Delft Hydraulics (1987)	Bed level changes due to abstraction
TABS-2	2 d depth av	steady	Ackers-White	USACE, USA	Thomas & Heath (1983)	Design of training works
ODYSEE	2 d width av	steady	van Rijn	DHL, Holland	Delft Hydraulics (1987)	Includes turbulence
TABS-3	3 d	unsteady	Ackers-White	USACE, USA	Thomas & Heath (1983)	Under development

Table 2 Recommended bend geometry for stable channel design

Reference	CURVE PARAMETERS Bend angle	R/B	Transition length	Comments
Bognar & Hanko (1987)	45 to 55	8 to 13.5		
Jansen et al (1979)	> 50		1-2B	
Kinori & Mevorach (1984)		7 to 8		Large/medium streams
Kinori & Mevorach (1984)		10 to 12		Small streams

Table 3 Orientation of groynes to upstream bank

Reference	Angle	Comment
UNCAFE, 1953	60 to 80	For bank protection
Jansen et al, 1979	90	Shortest and therefore cheapest
Garg et al, 1980	75 to 85	Indian practice
Copeland, 1983	90	Most effective bank protection
Bognar & Hanko, 1987	115	Hungarian practice

Table 4 Recommended groyne spacing for bank protection

Reference	Bank	Spacing	Comment
Grant, 1948	Concave	$3L_t$	
UNCAFE, 1953	Concave	$1L_t$	General practice
UNCAFE, 1953	Convex	2 to $2.5L_t$	General practice
Richardson & Simons, 1973	Concave	4 to $6L_t$	Bank may need riprap
Neill, 1973	Either	2 to $4L_t$	Four or more
Jansen et al, 1979	Either	$<0.6d^{4/3}/2gn^2$	Model tests at DHL
Los Angeles District, 1980	Straight	$2L_t$	Embankment should be
Los Angeles District, 1980	Concave	$1.5L_t$	protected with
Los Angeles District, 1980	Convex	$2.5L_t$	riprap
Garg et al, 1980	Either	3 to $4L_t$	Upstream orientation
Maccaferri, 1980	Concave	$4L_t$	Gabions
Maccaferri, 1980	Convex	$6L_t$	Gabions
Copeland, 1983	Concave	upto $3L_t$	Bank may need riprap
Bognar & Hanko, 1987	Either	$1.2L_t$	Maximum siltation
Alvarez, 1989	Straight	4 to $6L_t$	For irregular curves
Alvarez, 1989	Concave	2.5 to $4L_t$	spacing found
			graphically
Salikov, 1987	Concave	7 to $10L_t$	

Table 5 Length of guide banks

Reference	Upsteam	Downsteam	Comment
Spring, 1903	1 to 1.1B	0.1 to 0.2B	Based on Indian experience
Blench, 1969	0.75B	0.25B	Regime theory
Neill, 1973	0.75B	0.25B	Meandering rivers
Andreev as reported by Neill, 1973	Function of discharge	0.33 of upstream	Flood plain rivers well defined channels
Garg et al, 1980	Function of meander pattern	0.25 to 0.4B	Length chosen to protect embankment from worst possible meander loop

FIGURES.

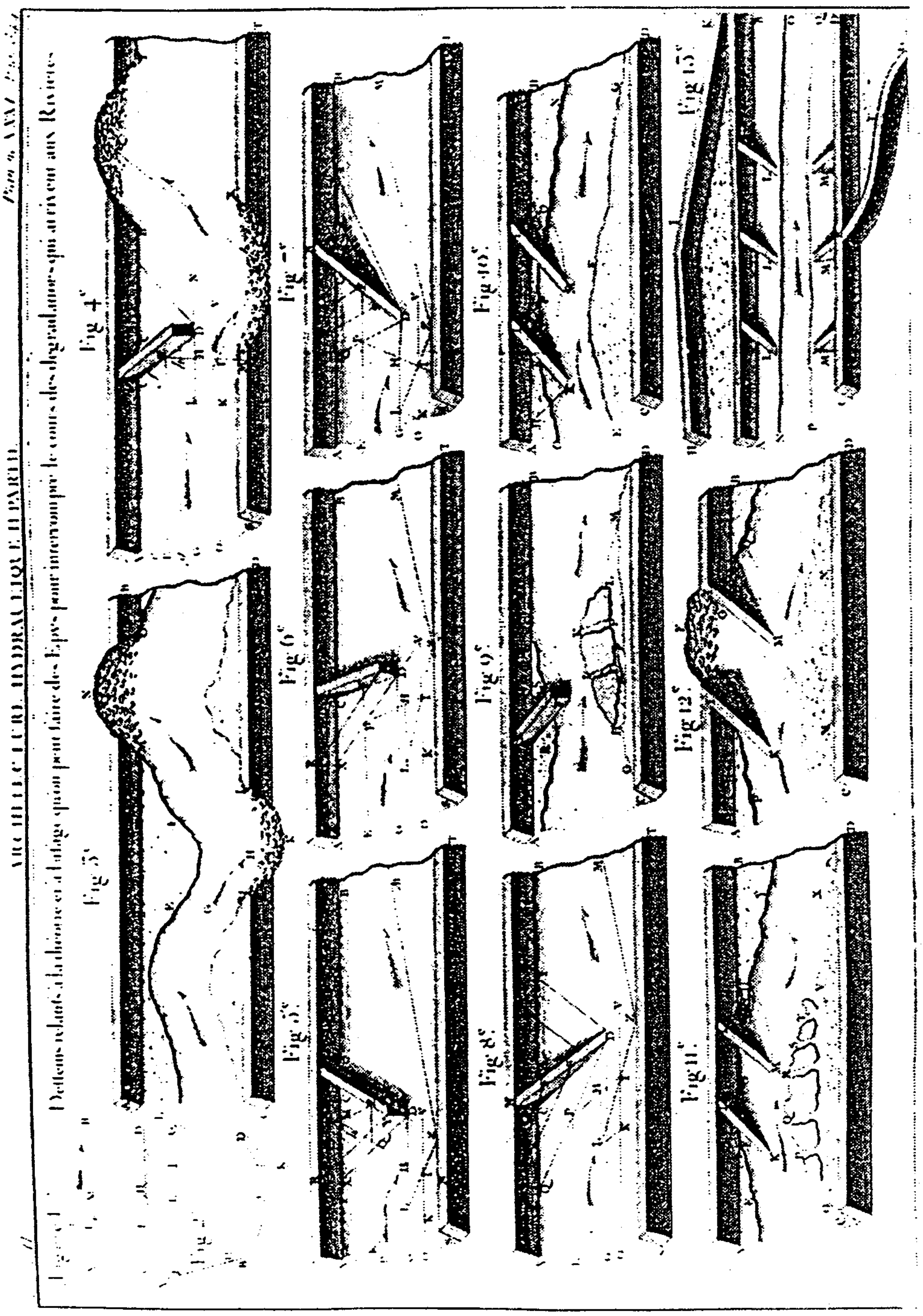


Fig 1 The classical understanding of groynes
(after Belidor, 1737)

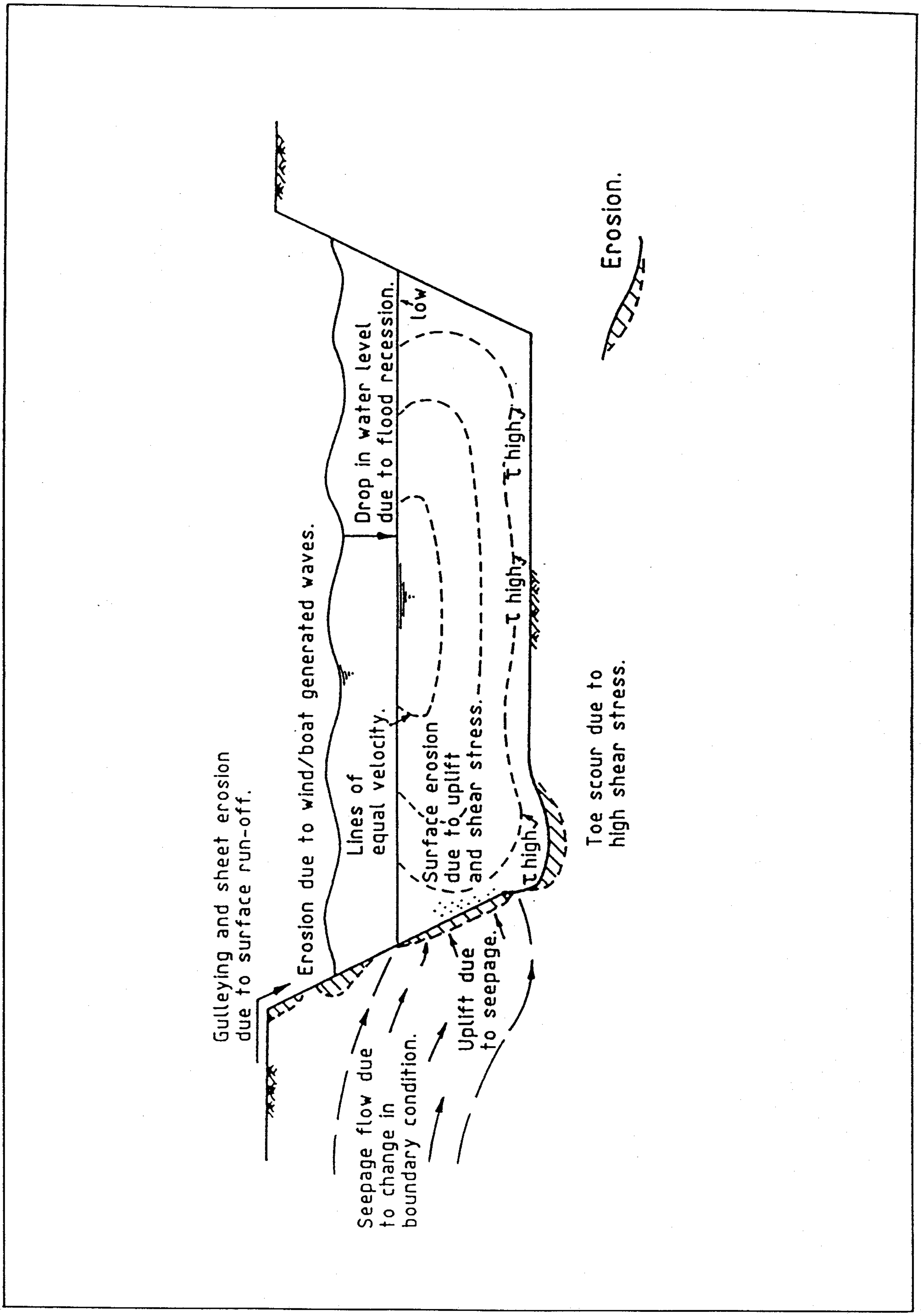


Fig 2 Hydraulic processes causing bank and bed erosion (after Brandon, 1989)

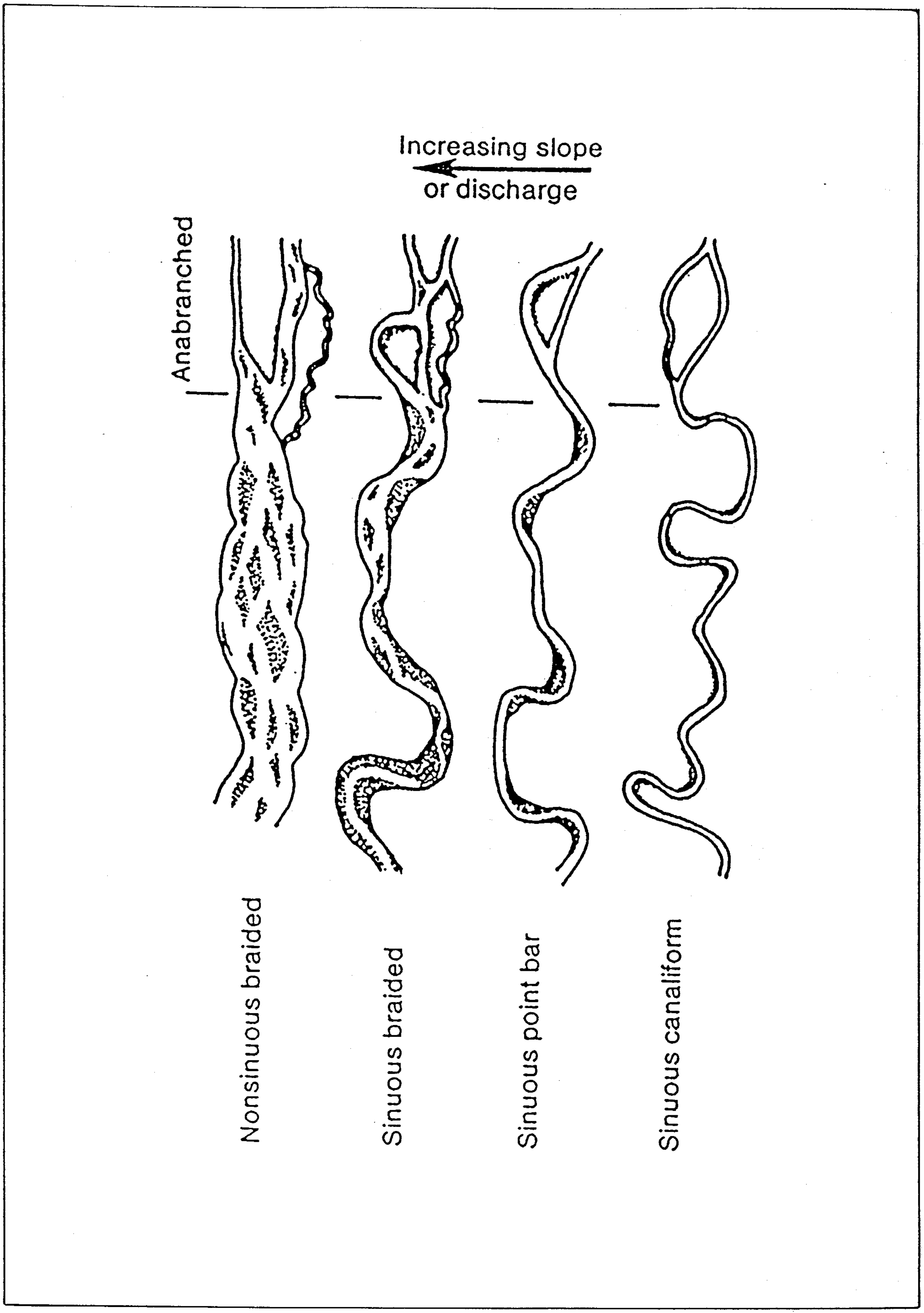


Fig 3 Classification of river planforms (after Brice, 1983)

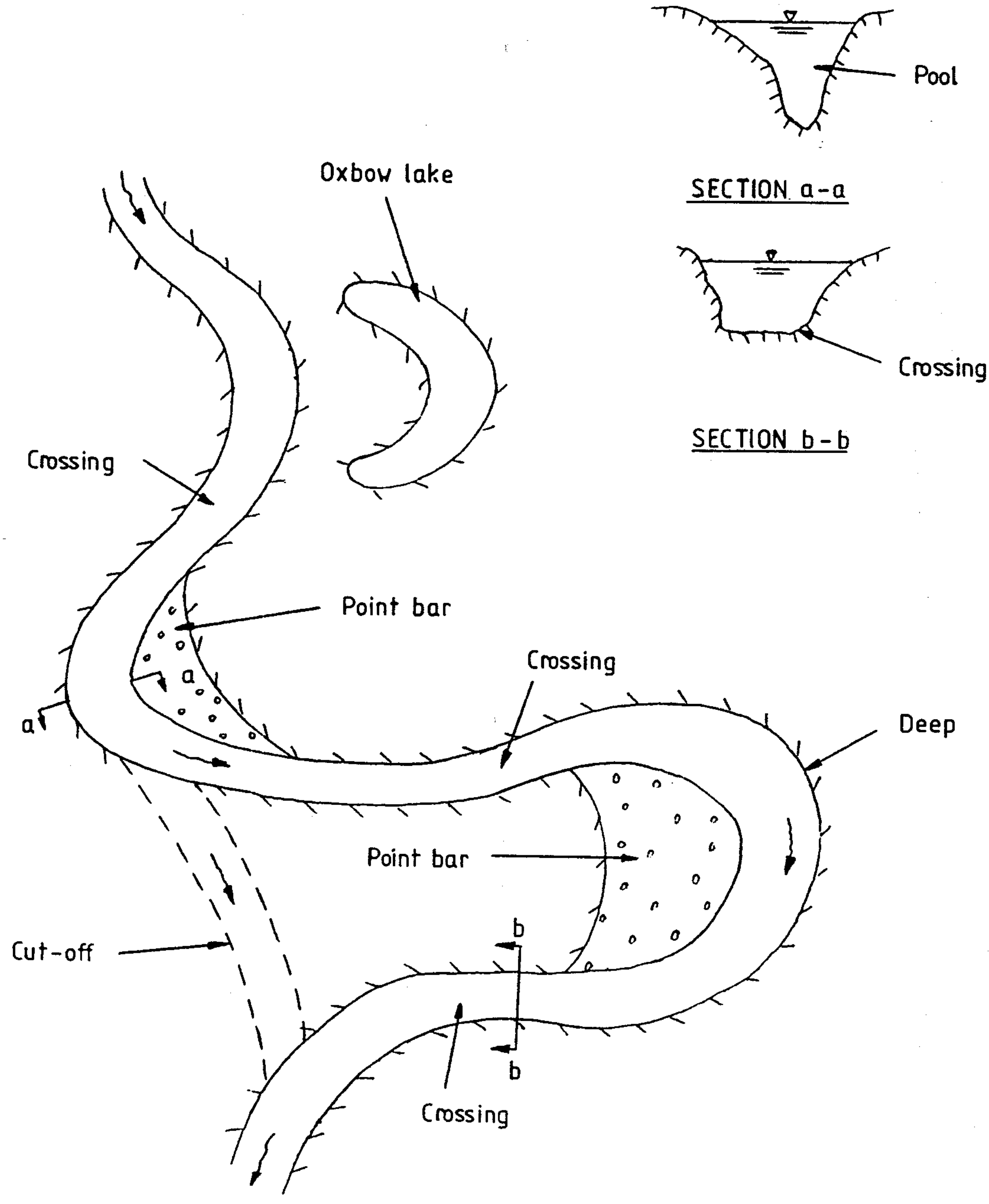


Fig 4 Features of natural meandering channels (after Kinori and Mevorach, 1984)

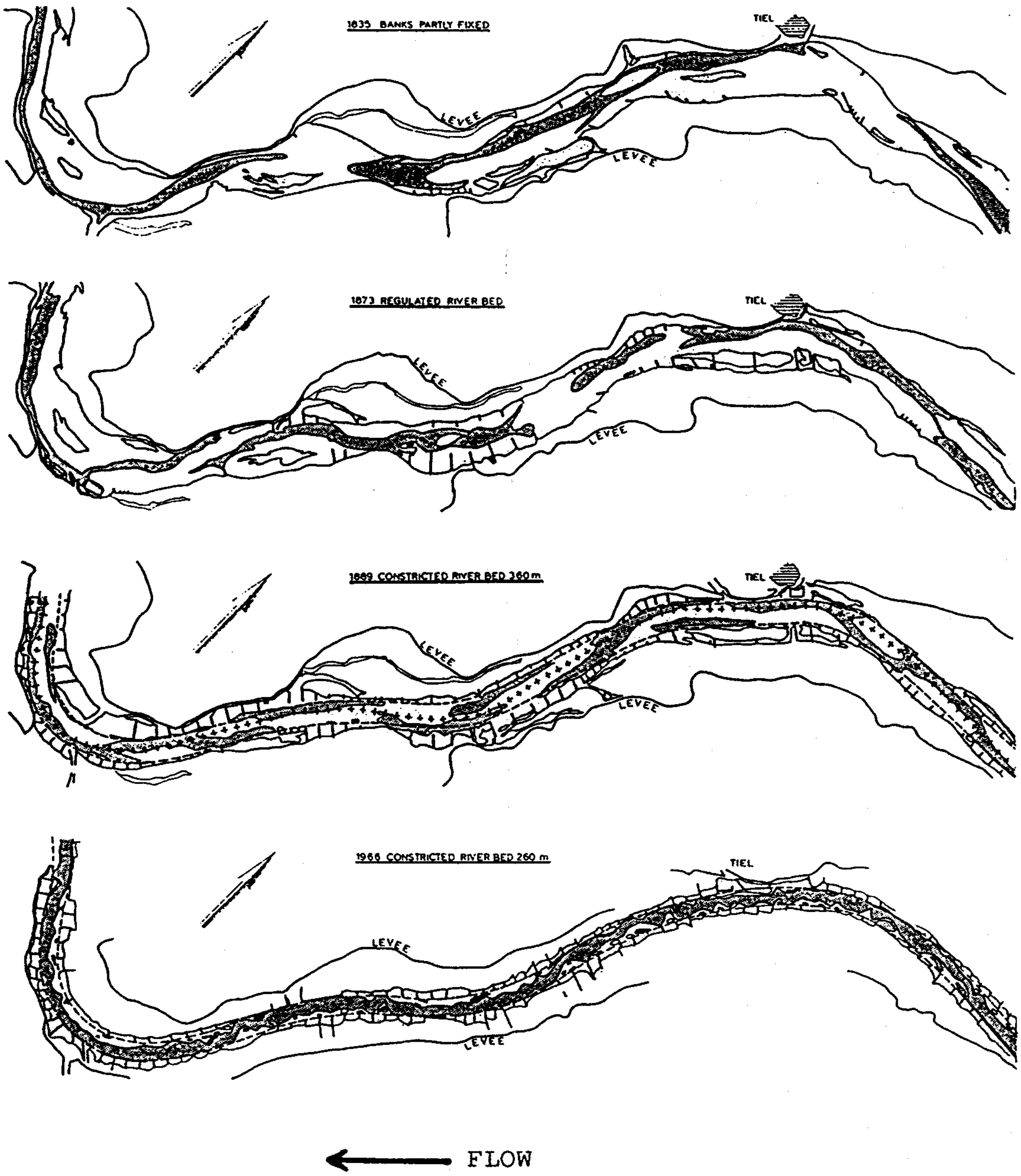


Fig 5 History of channelisation of the River Waal, Holland (after Jansen et al, 1979)

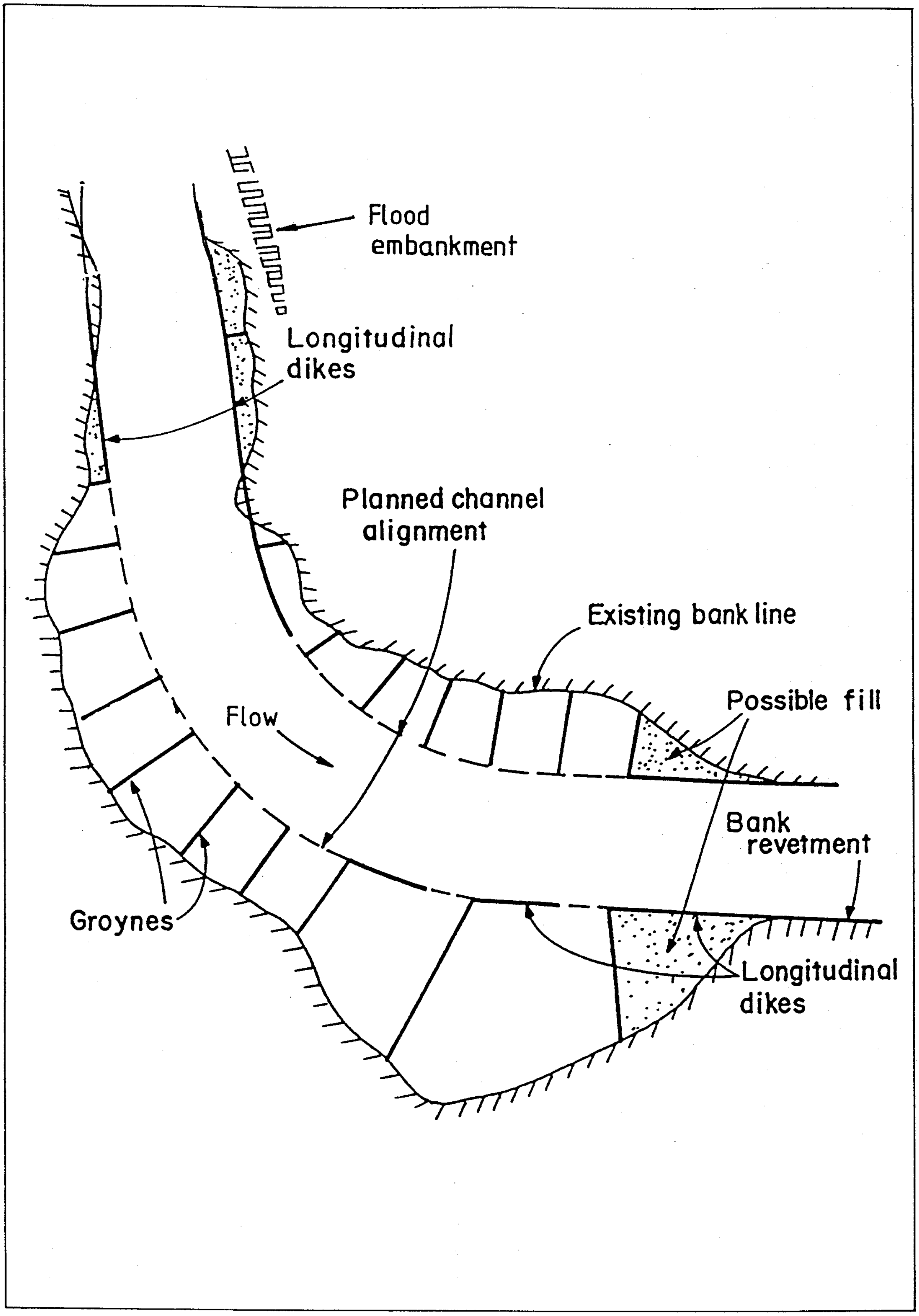


Fig 6 Typical layout of training works for channelisation (after Kinori and Mevorach, 1984)

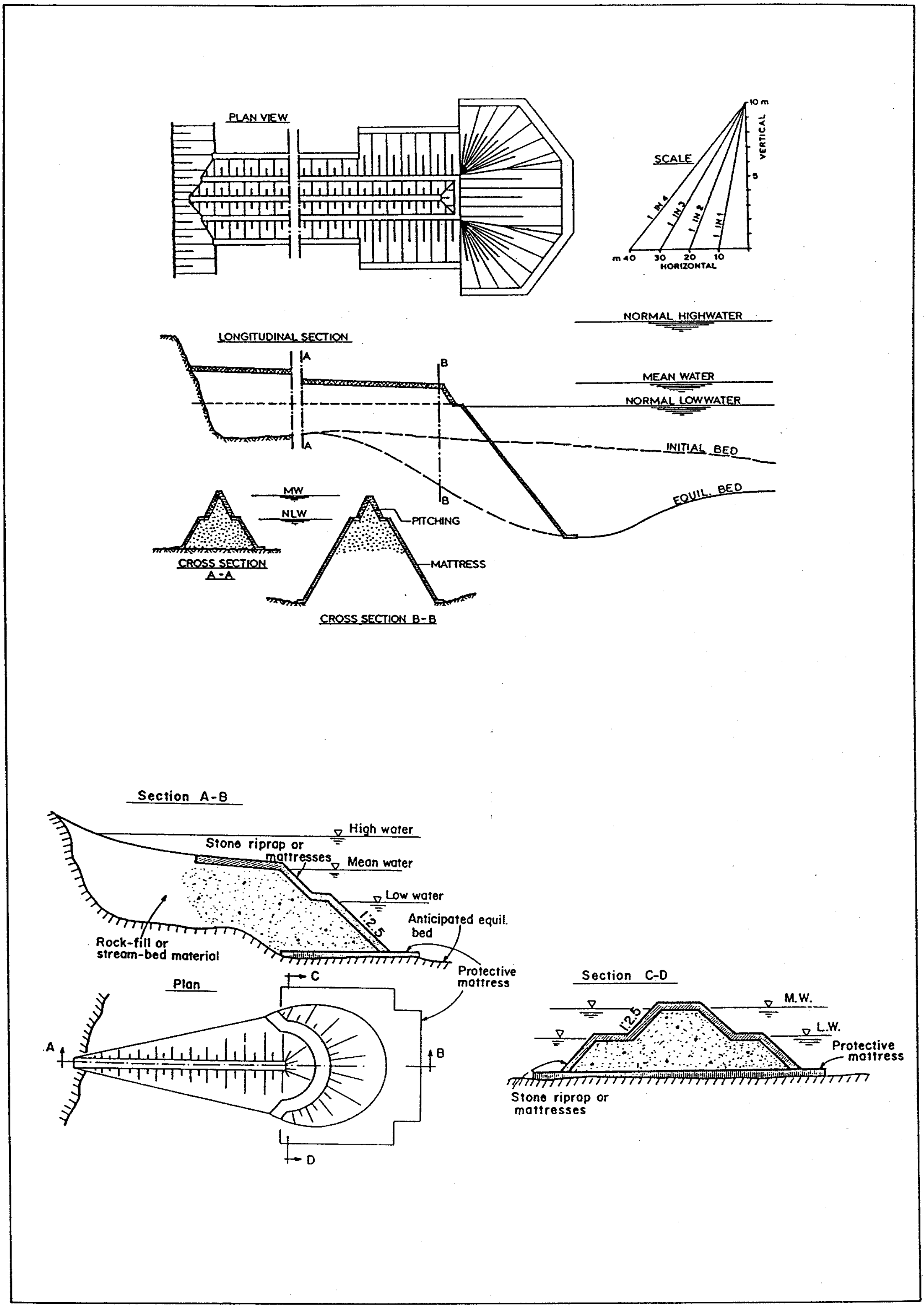


Fig 7 Typical impermeable groyne structures

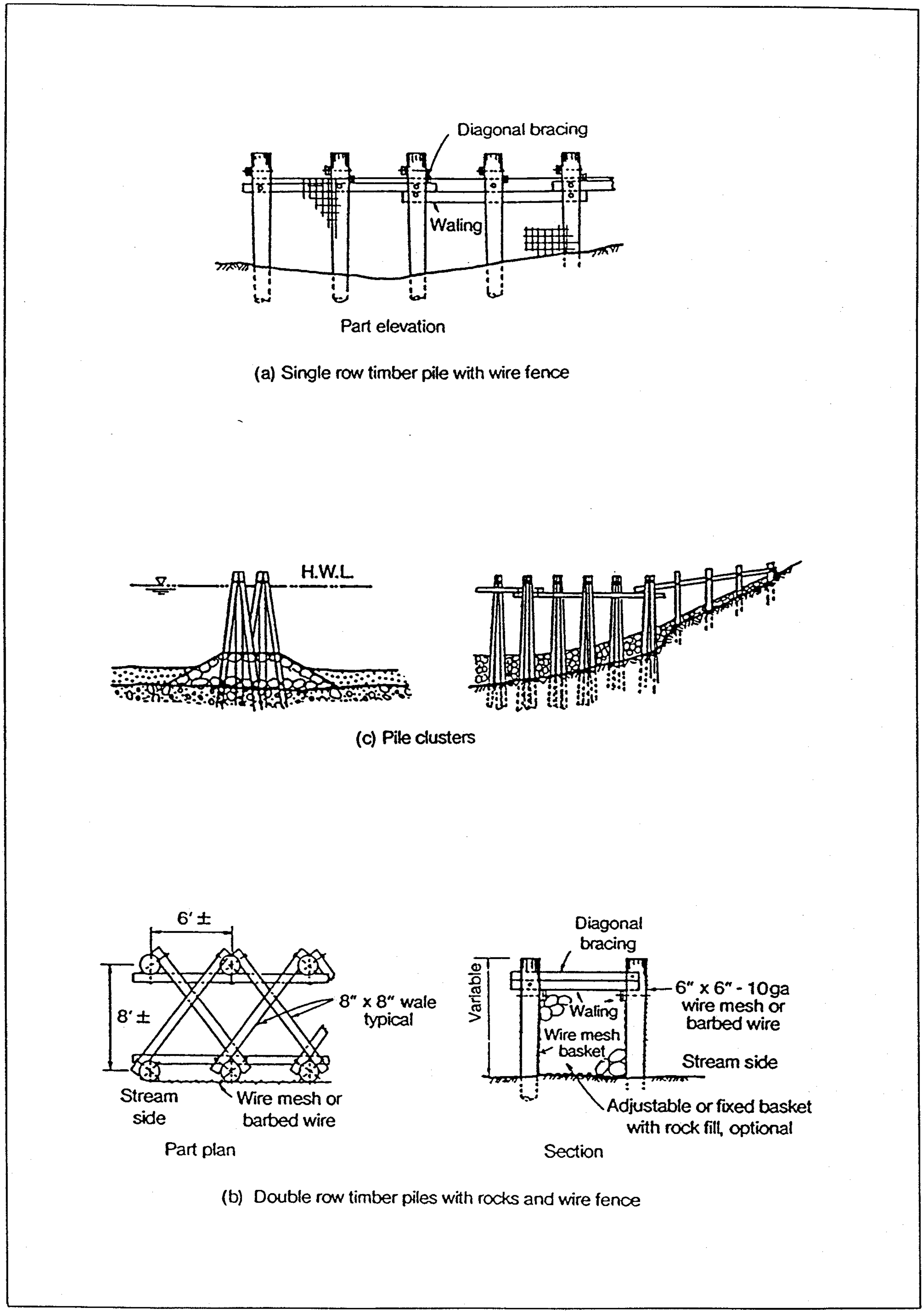


Fig 8 Permeable timber pile groynes (after Chang, 1988)

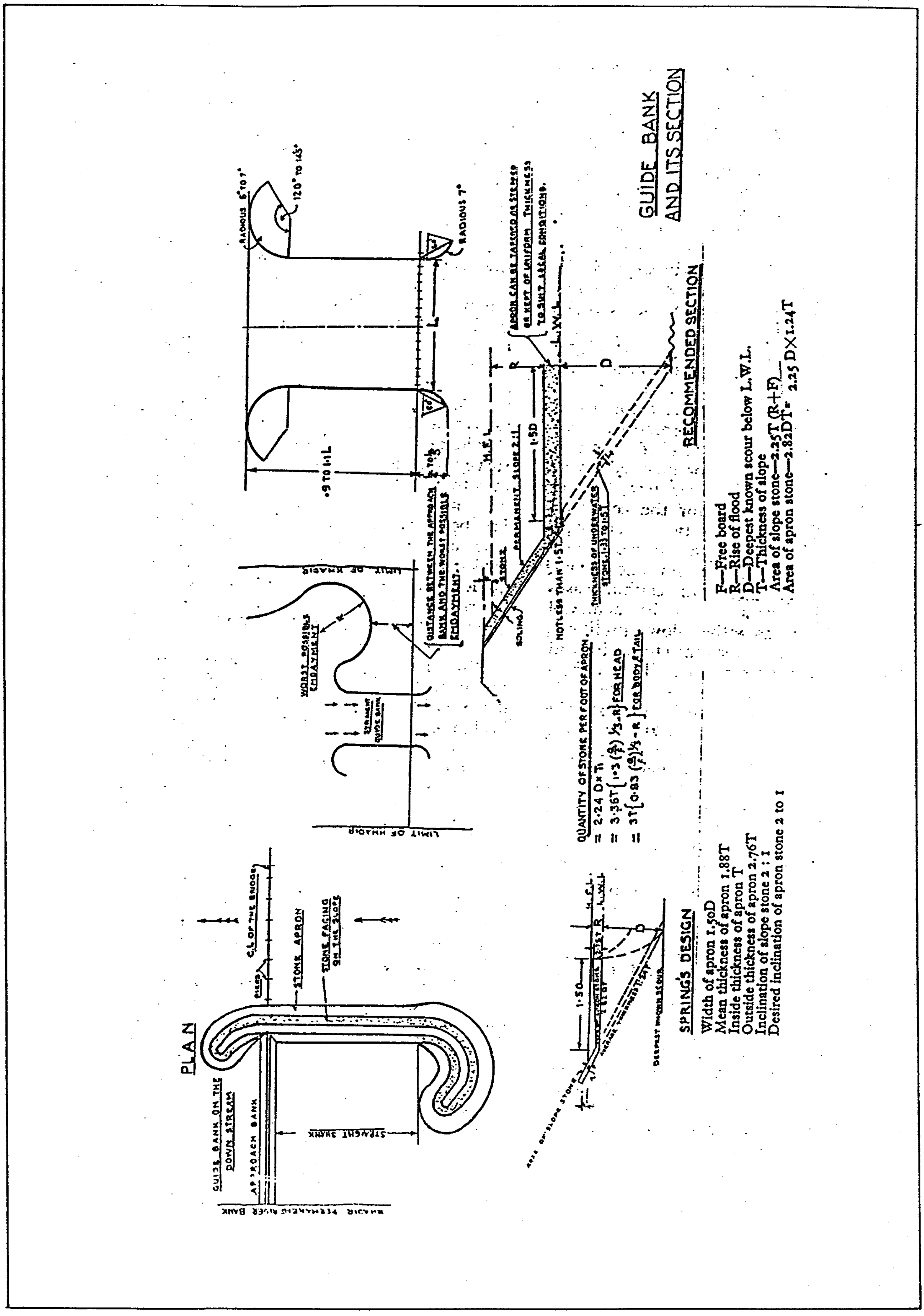


Fig 9 Typical design of guidebanks (after Sethi,1960)

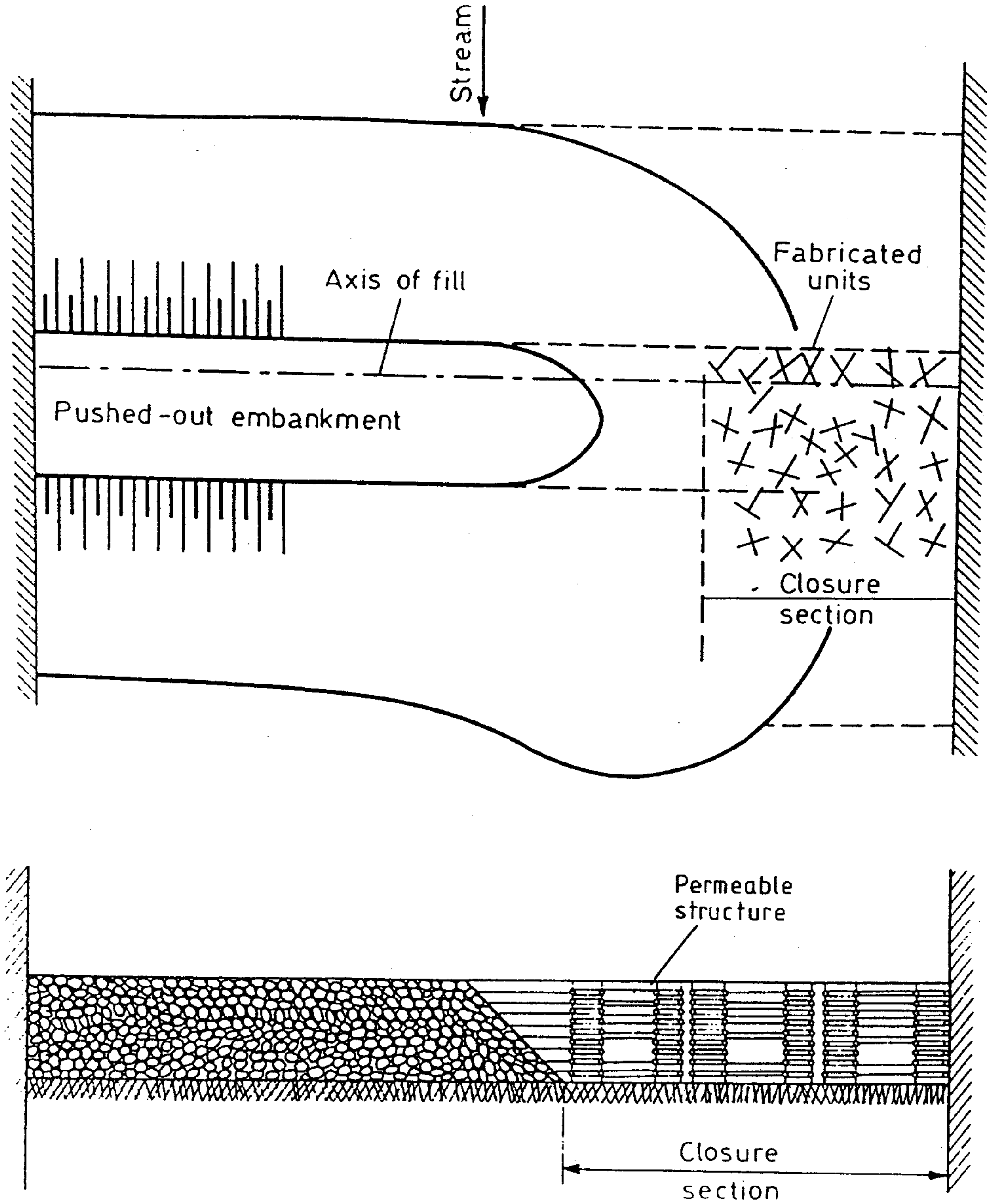
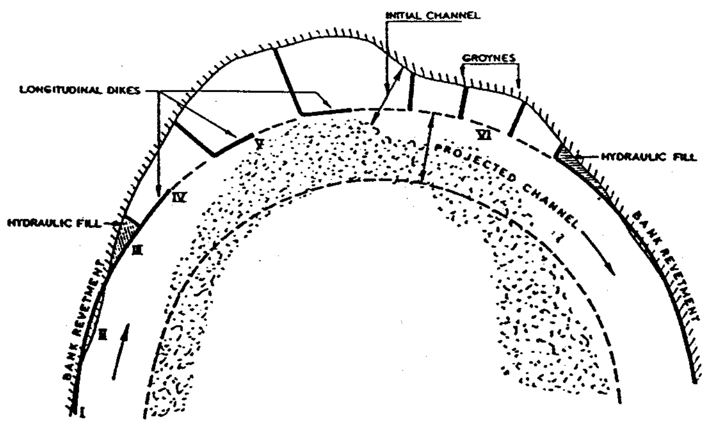
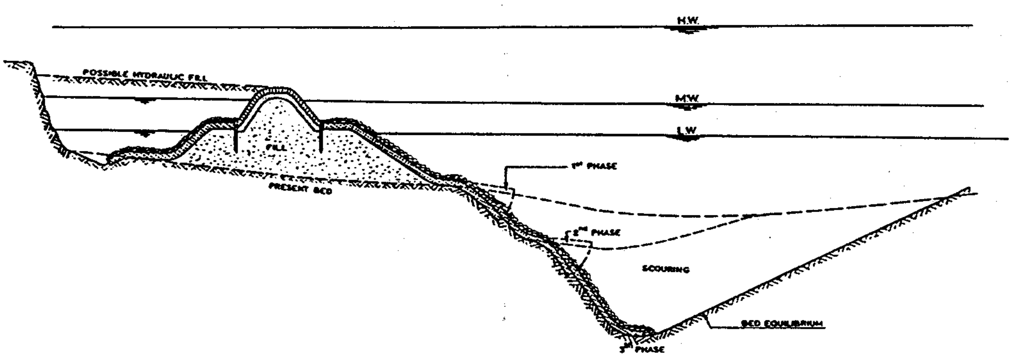


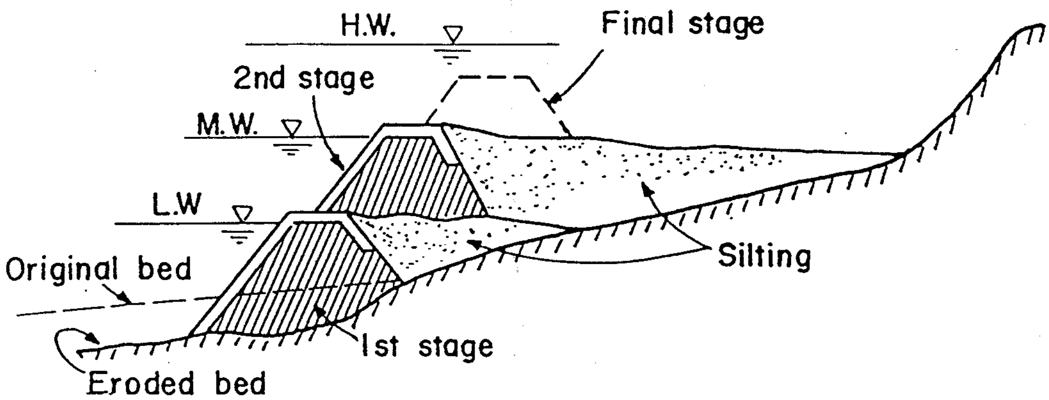
Fig 10 Combined method of channel cut-off construction (after Izbash and Khaldre, 1970)



a. The use of longitudinal dikes for channelisation

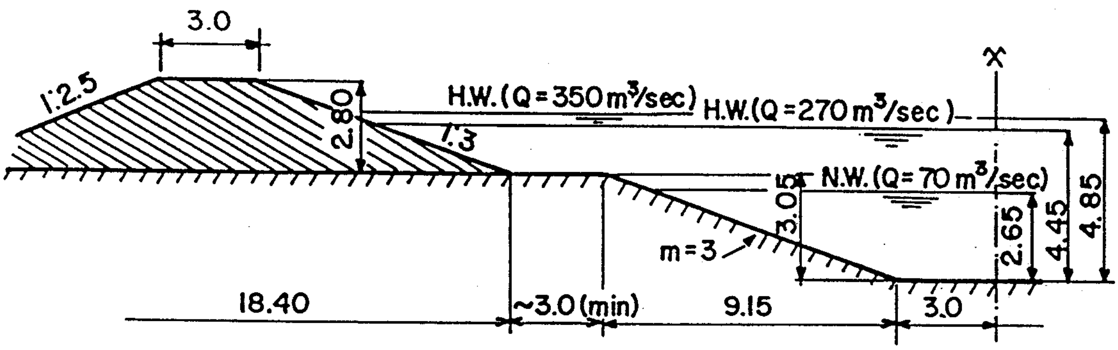


b. Typical cross-section of longitudinal dike

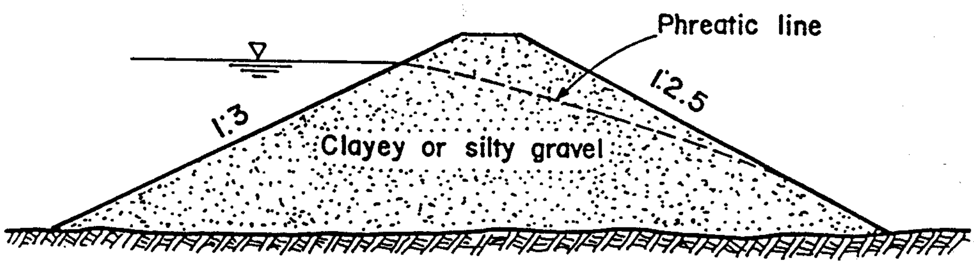


c. Staged construction of longitudinal dike

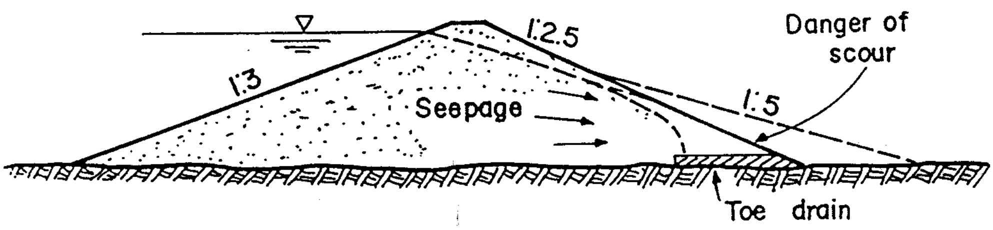
Fig 11 Typical longitudinal dikes (after Jansen et al, 1979)



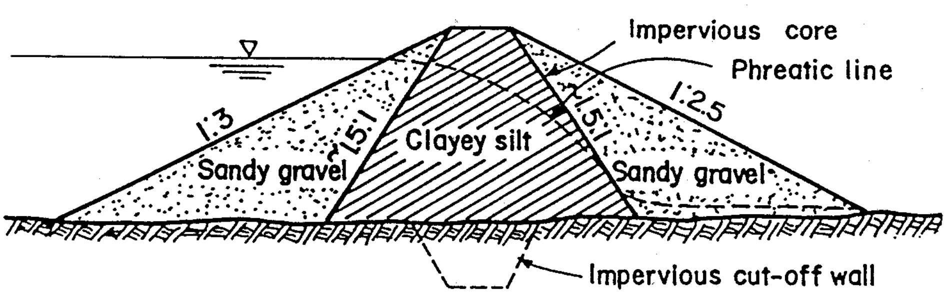
a. Typical flood embanked section



b. Typical homogeneous embankment.

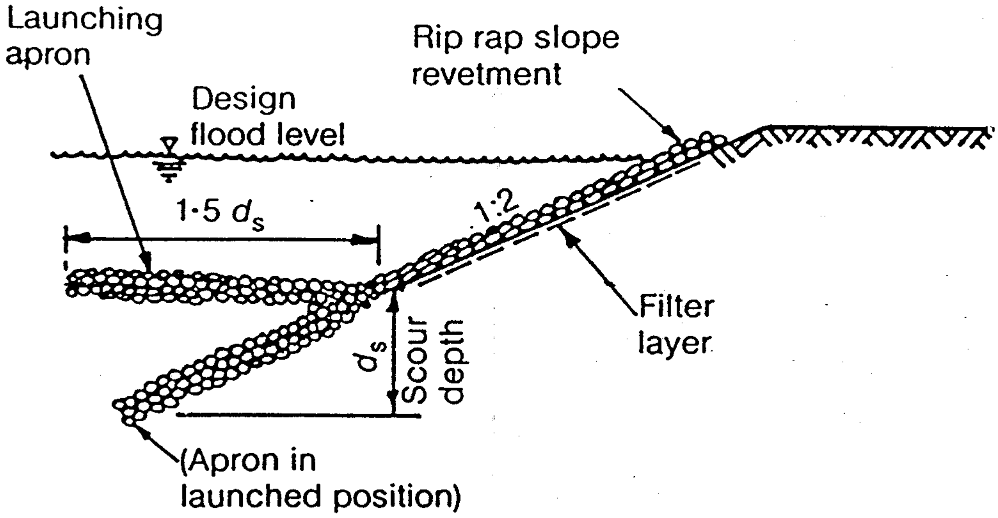


c. Typical pervious fill embankment with toe drain.

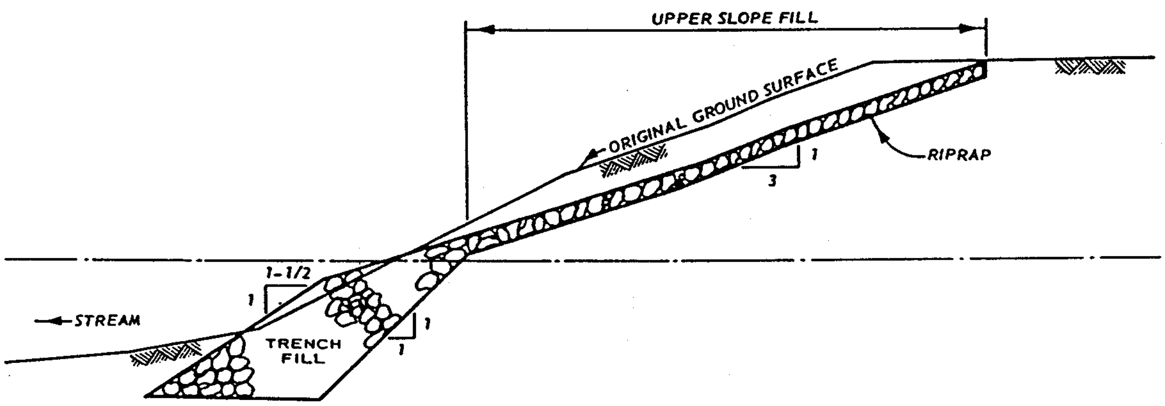


d. Typical zoned embankment.

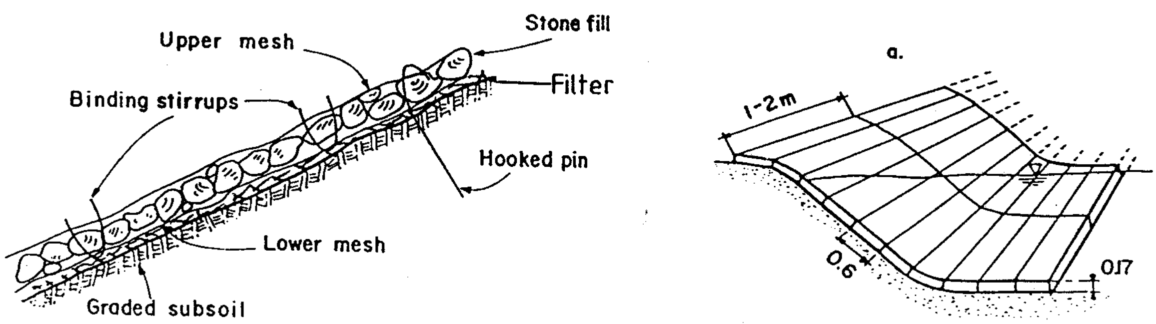
Fig 12 Typical flood embankment cross-sections (after Kinori and Mevorach, 1984)



a. Rip rap revetment with launching apron

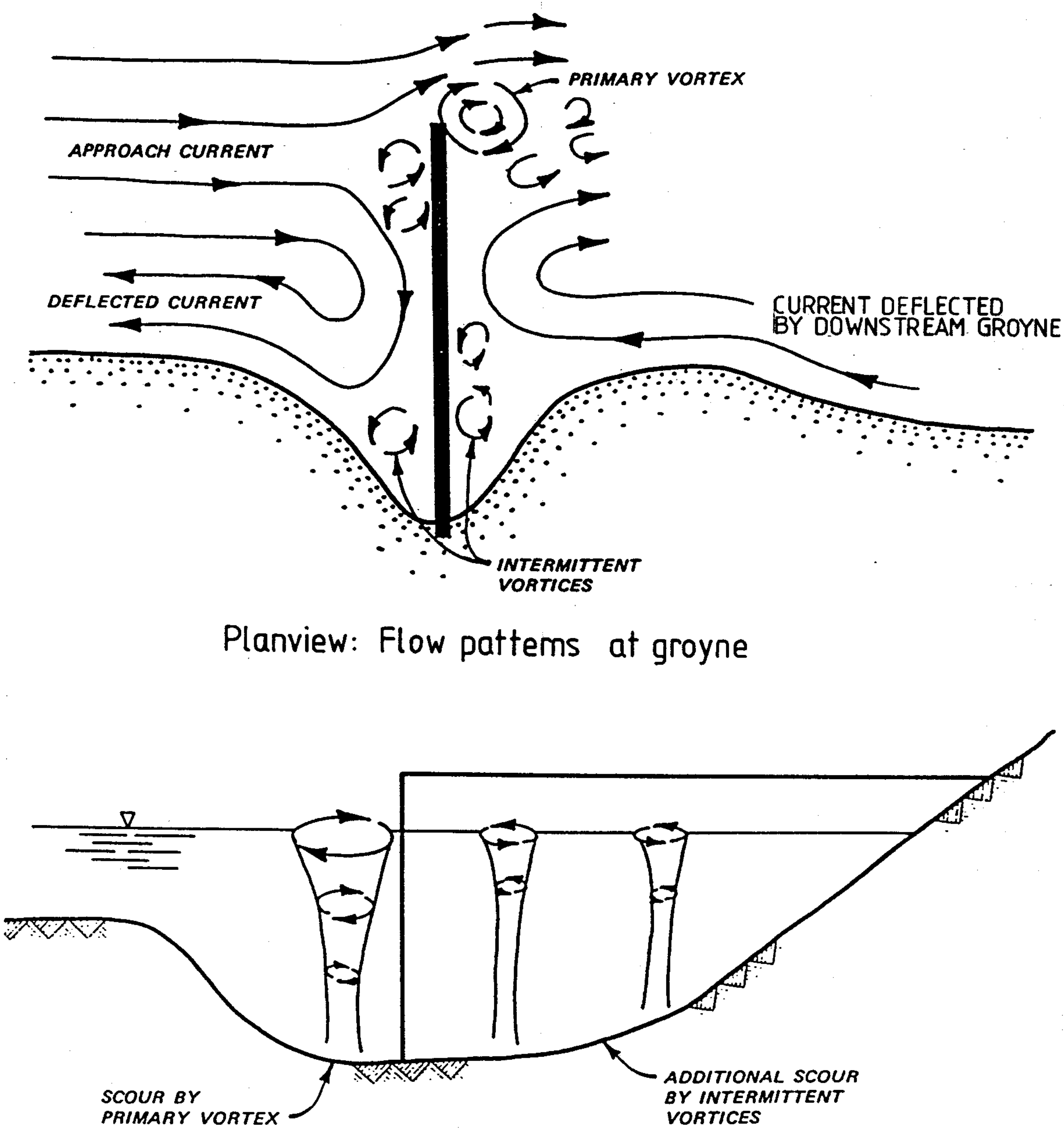


b. Trench fill revetment



c. Flexible stone matress

Fig 13 Examples of bank revetment



Planview: Flow patterns at groyne

Elevation: Scour hole profile along groyne

Fig 14 Local vortex systems around a groyne
(after Copeland, 1983)

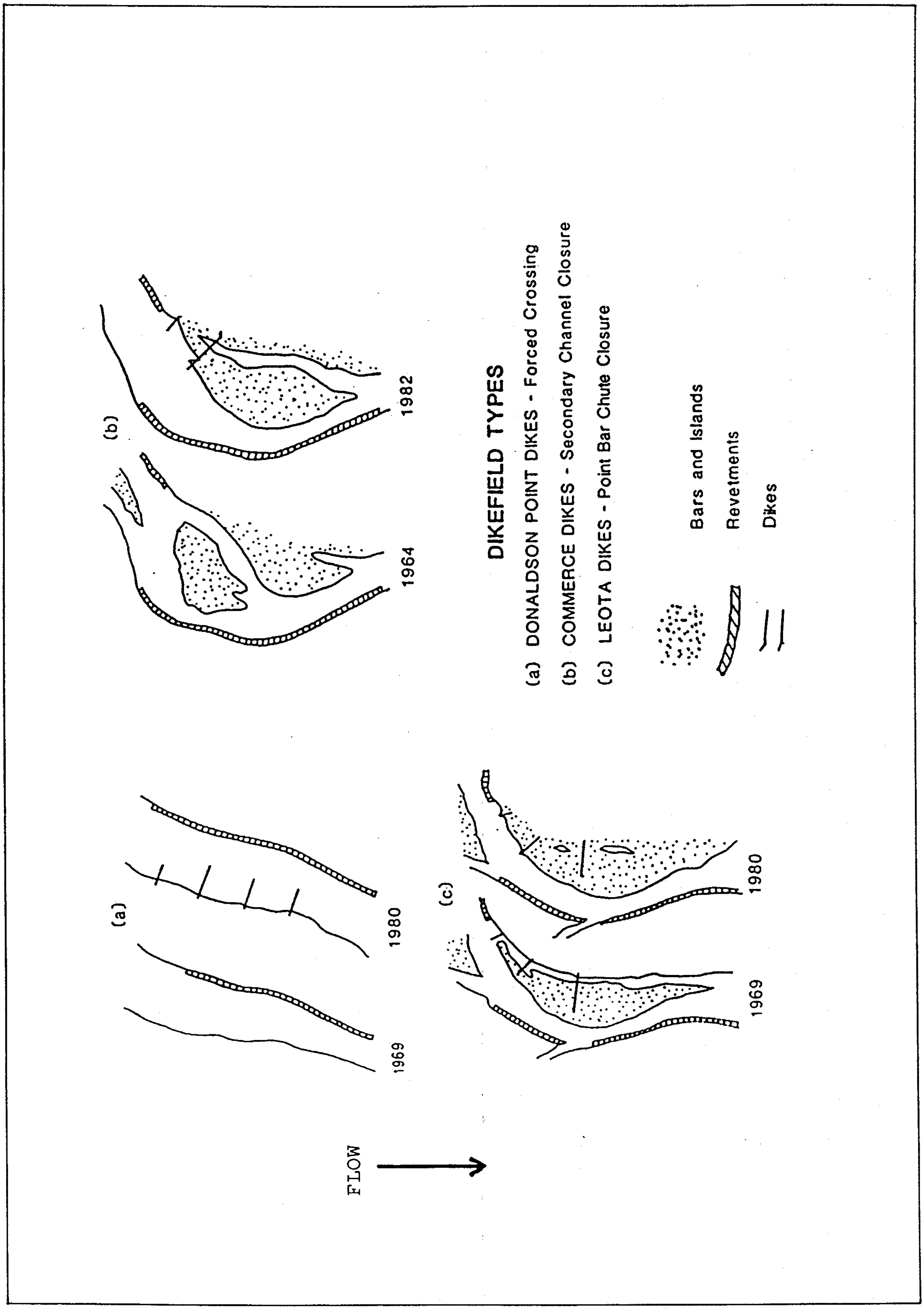


Fig 15 Effects of dikefields on the Lower Mississippi River (after Nunnally and Beverly, 1983)

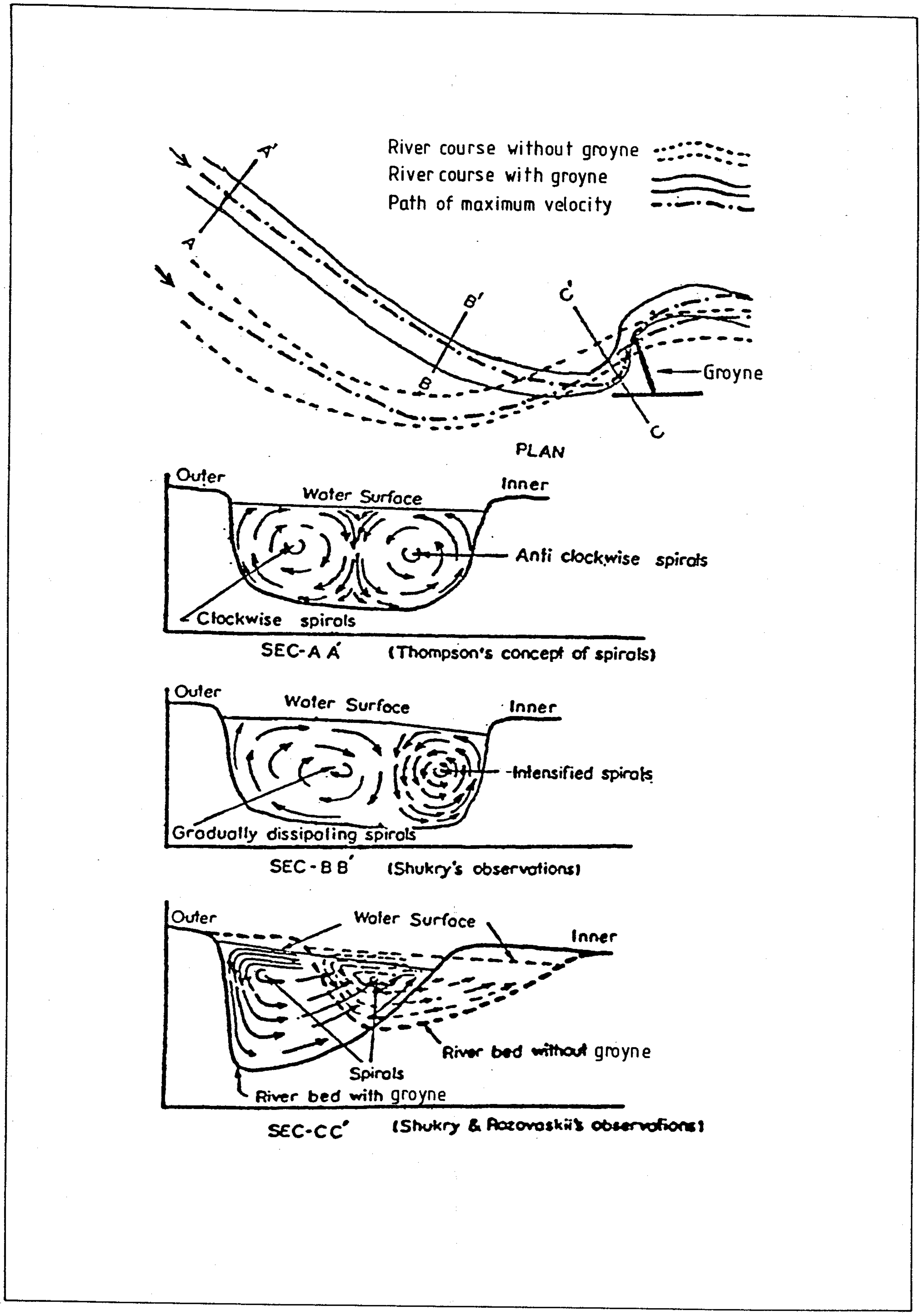


Fig 16 The planform response of a river to a single groyne (after Bhargava and Singh, 1981)

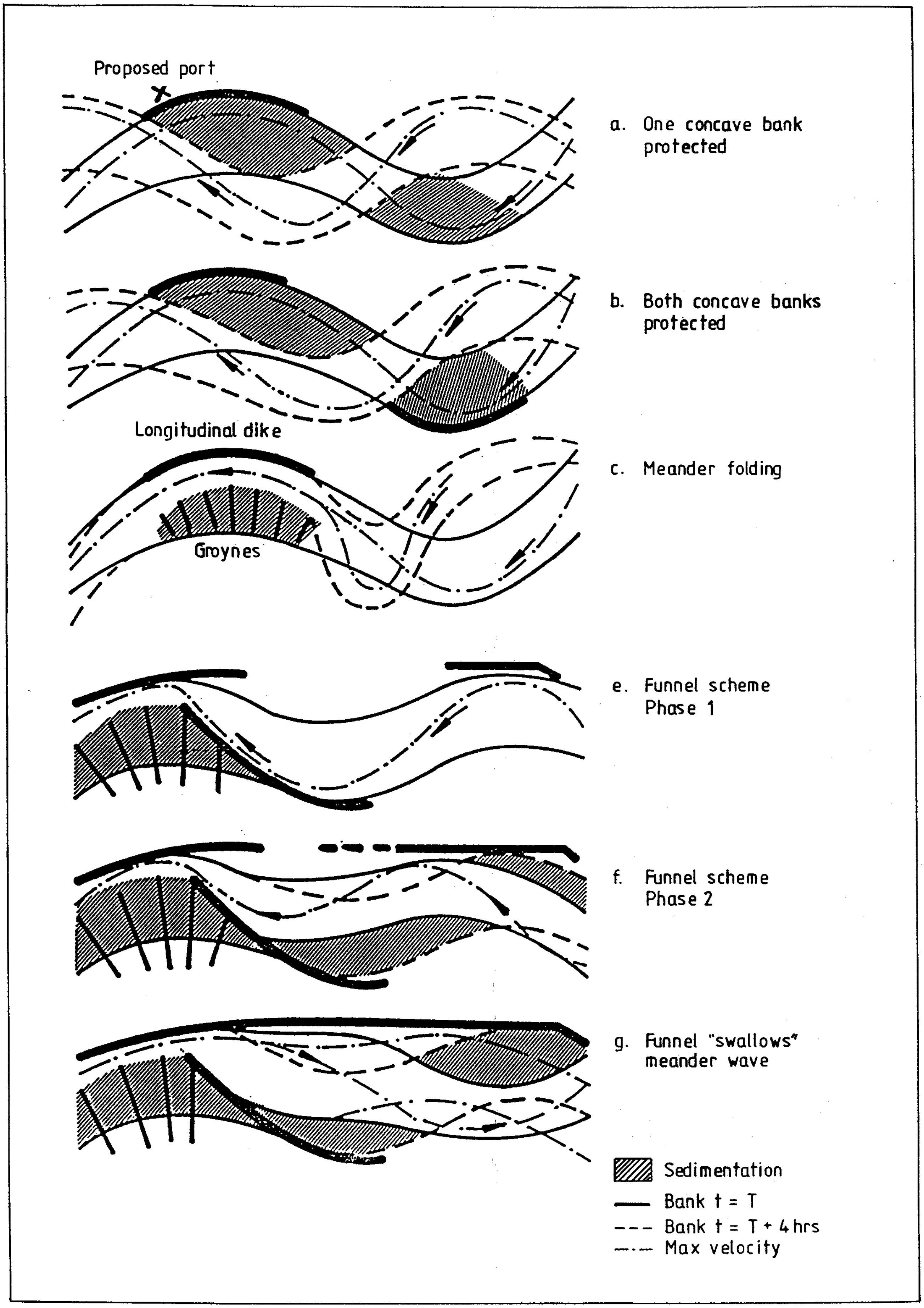


Fig 17 Physical model study of meander stabilisation (after Berlamont and Schiara,1983)

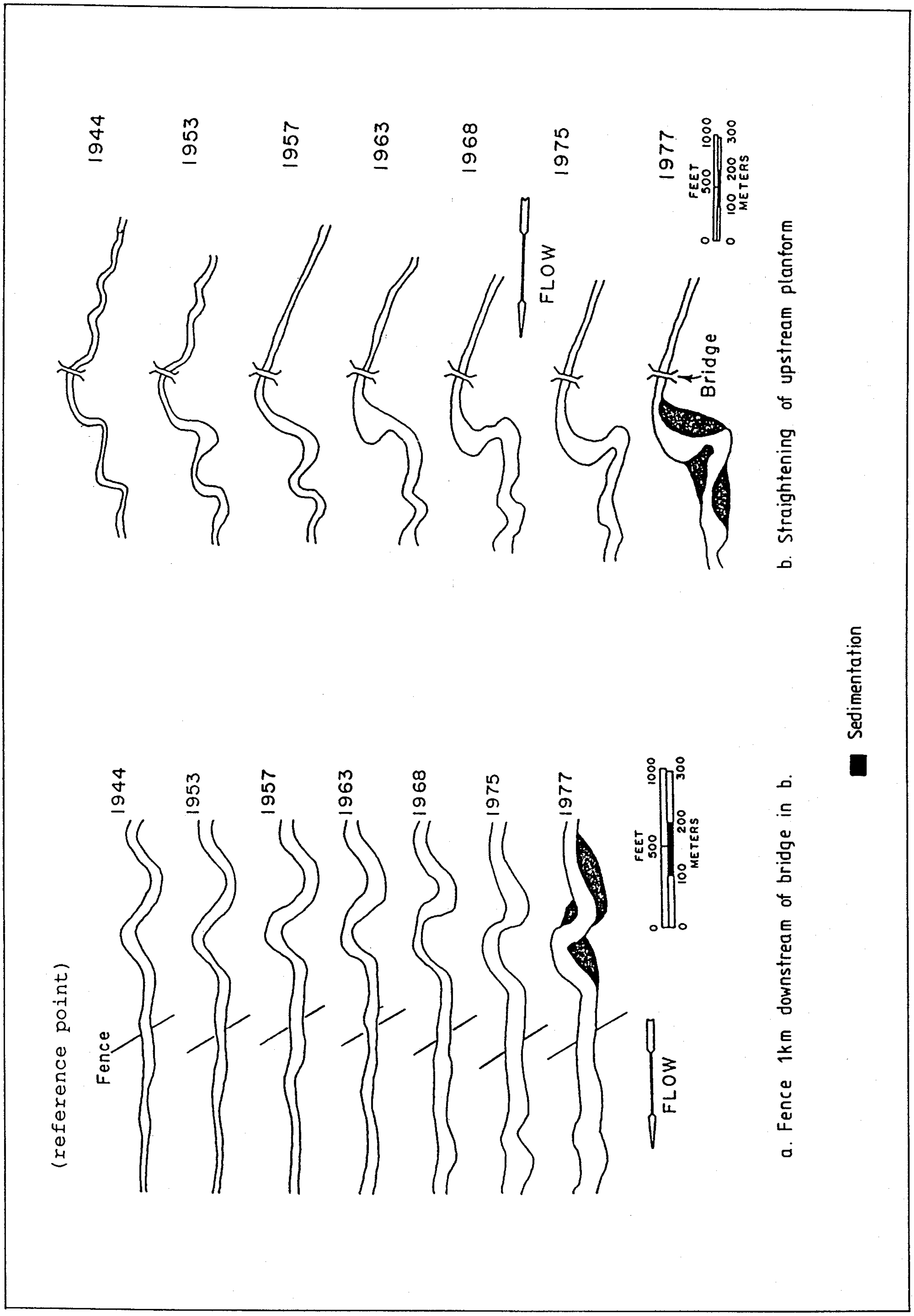


Fig 18 Historical planform change around structures on Lower Goodwin Creek (Grissinger and Murpley, 1983)

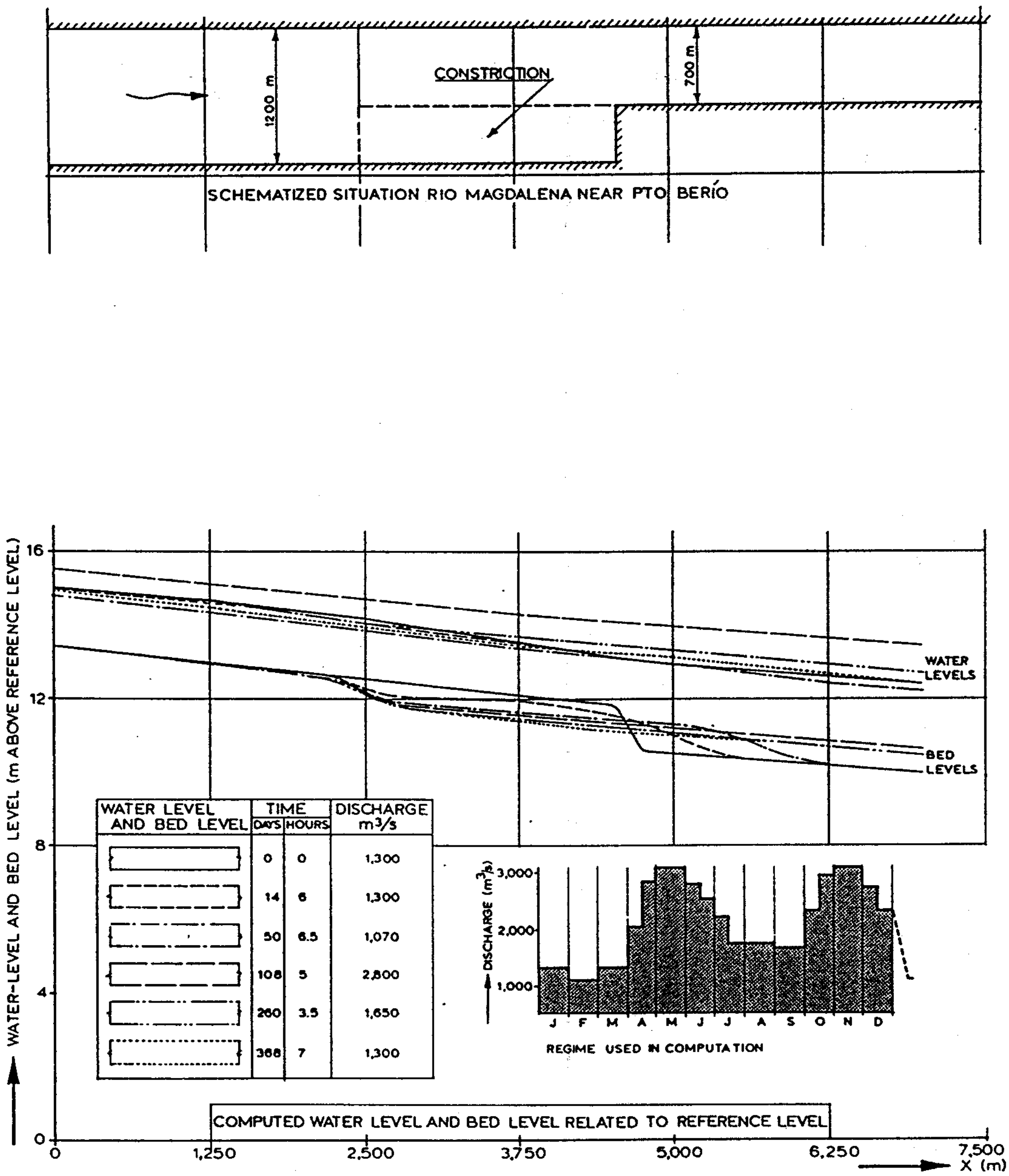


Fig 19 Morphological impact of channel constriction (after Jansen et al,1979)

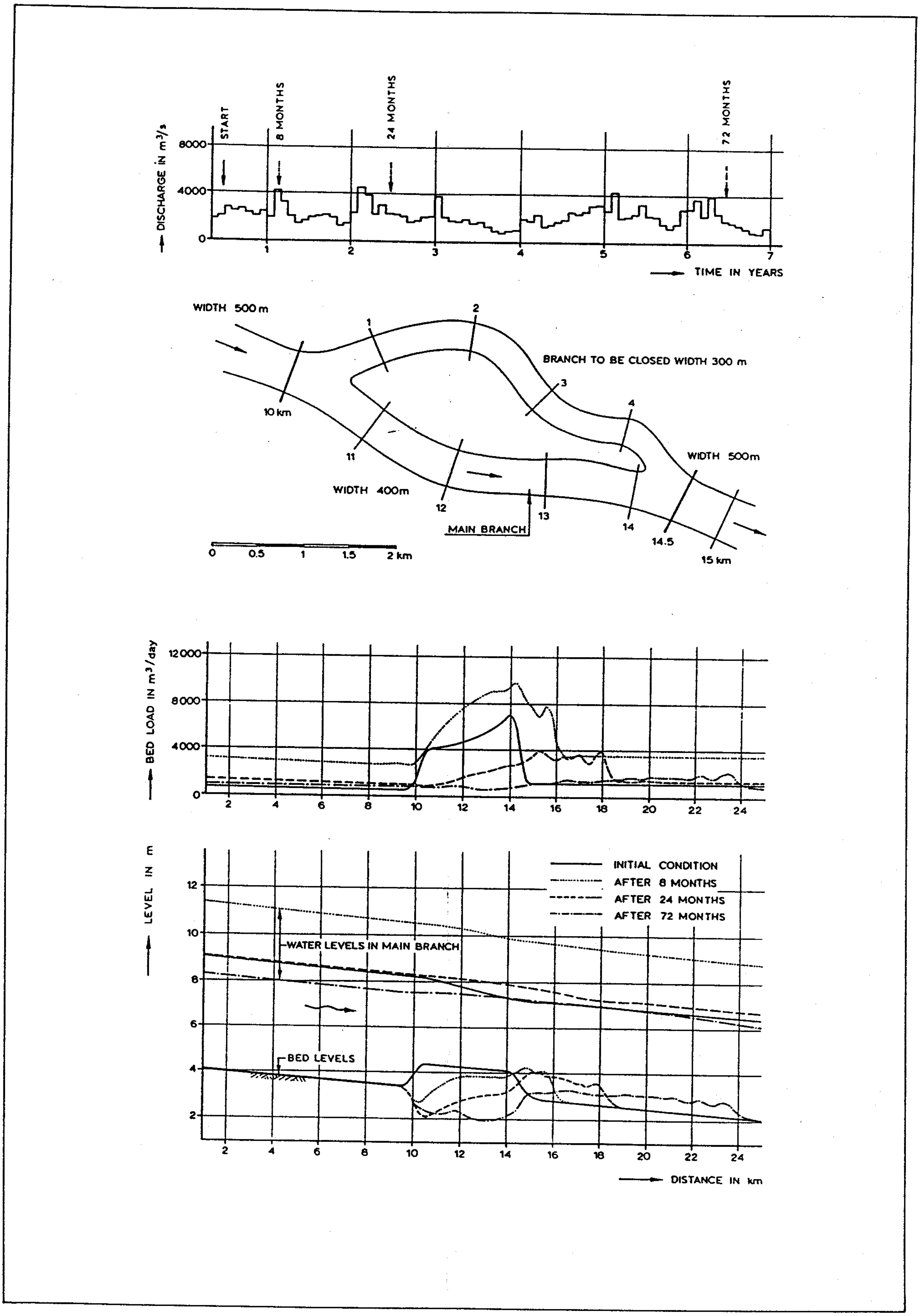


Fig 20 Morphological effects of closing a river branch (after Jansen et al, 1979)

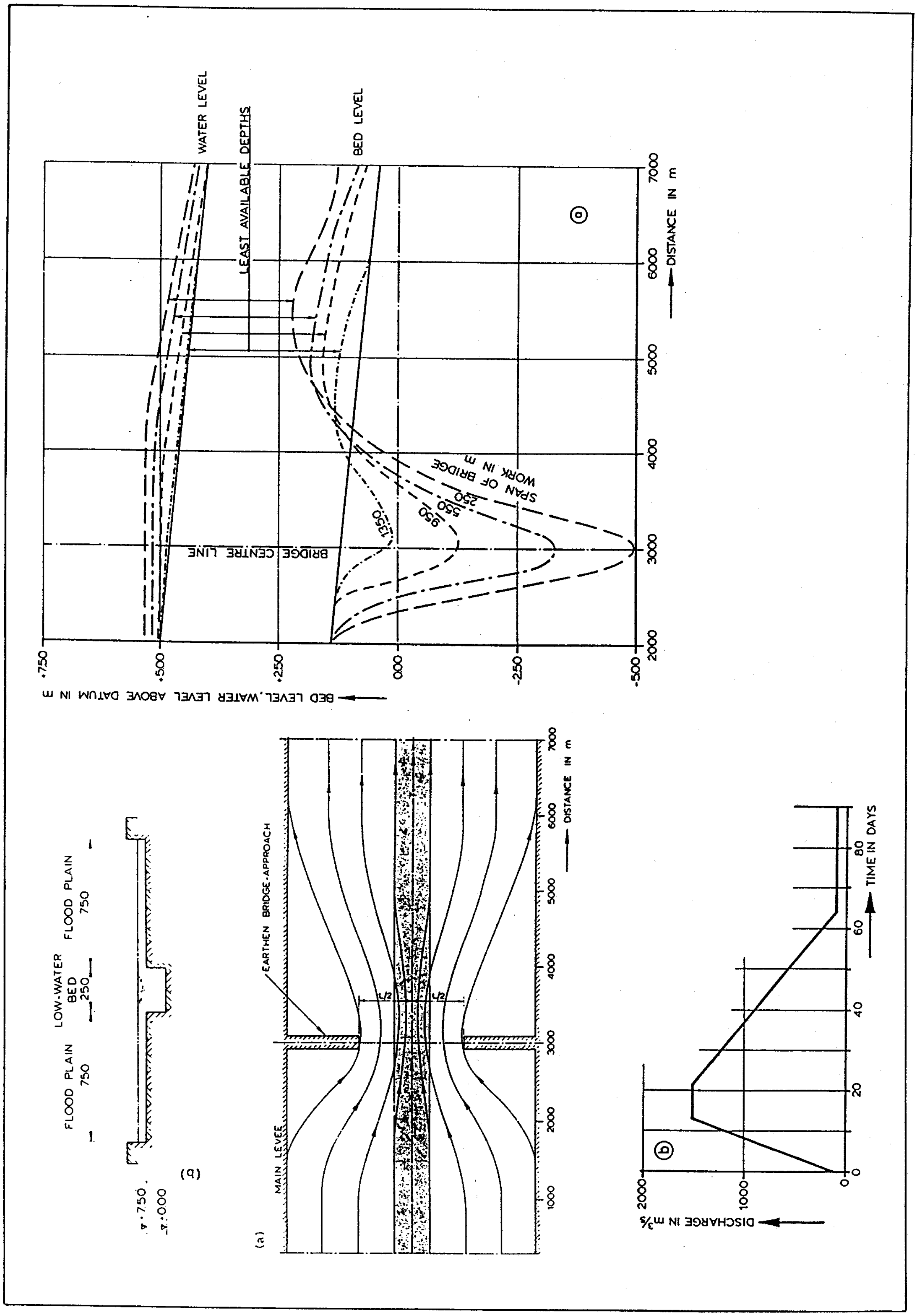


Fig 21 Morphological impact of channel constriction at bridge (after Jansen et al, 1979)

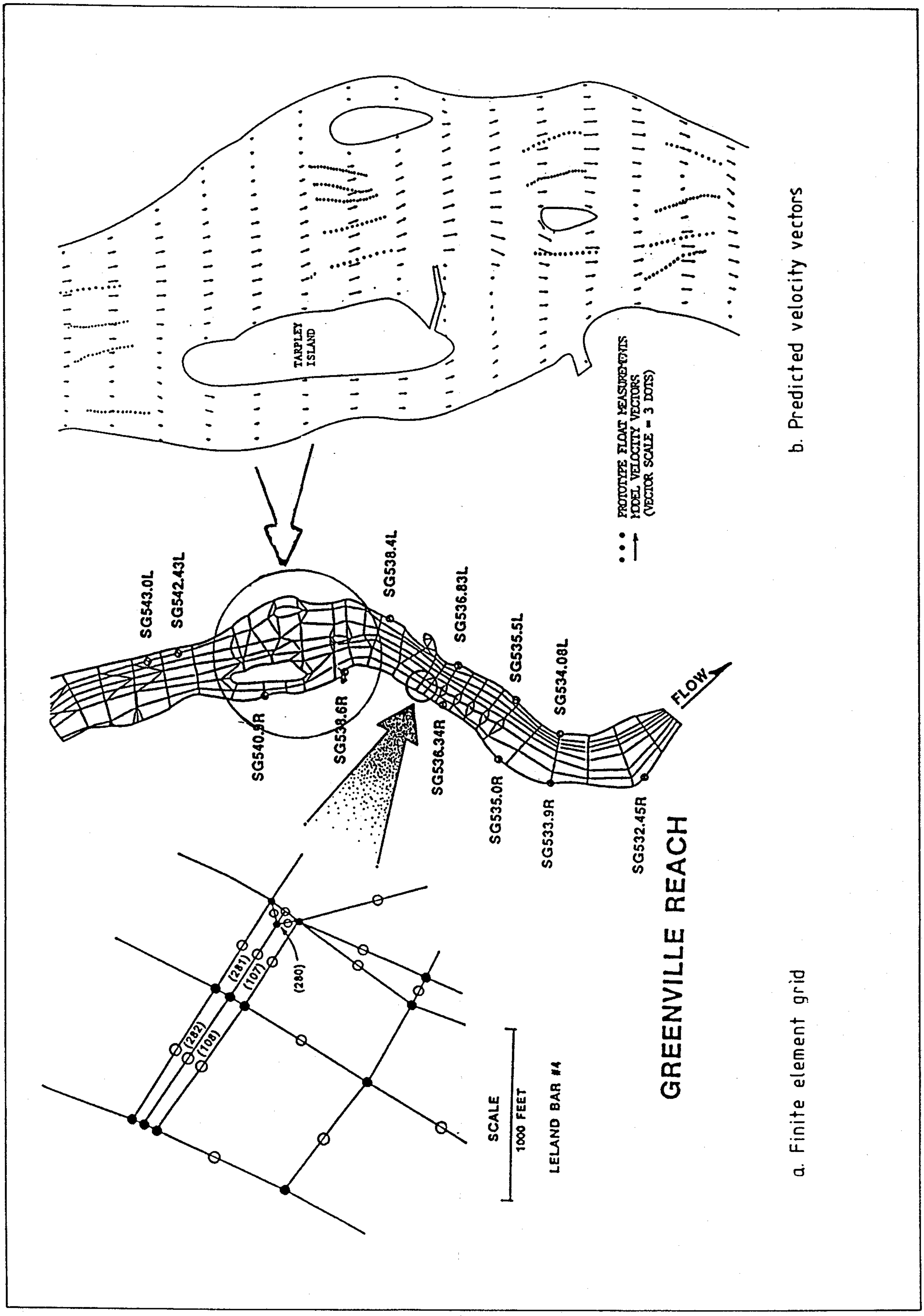


Fig 22 Application of TABS - 2 to the Mississippi River (after Thomas and Heath, 1983)

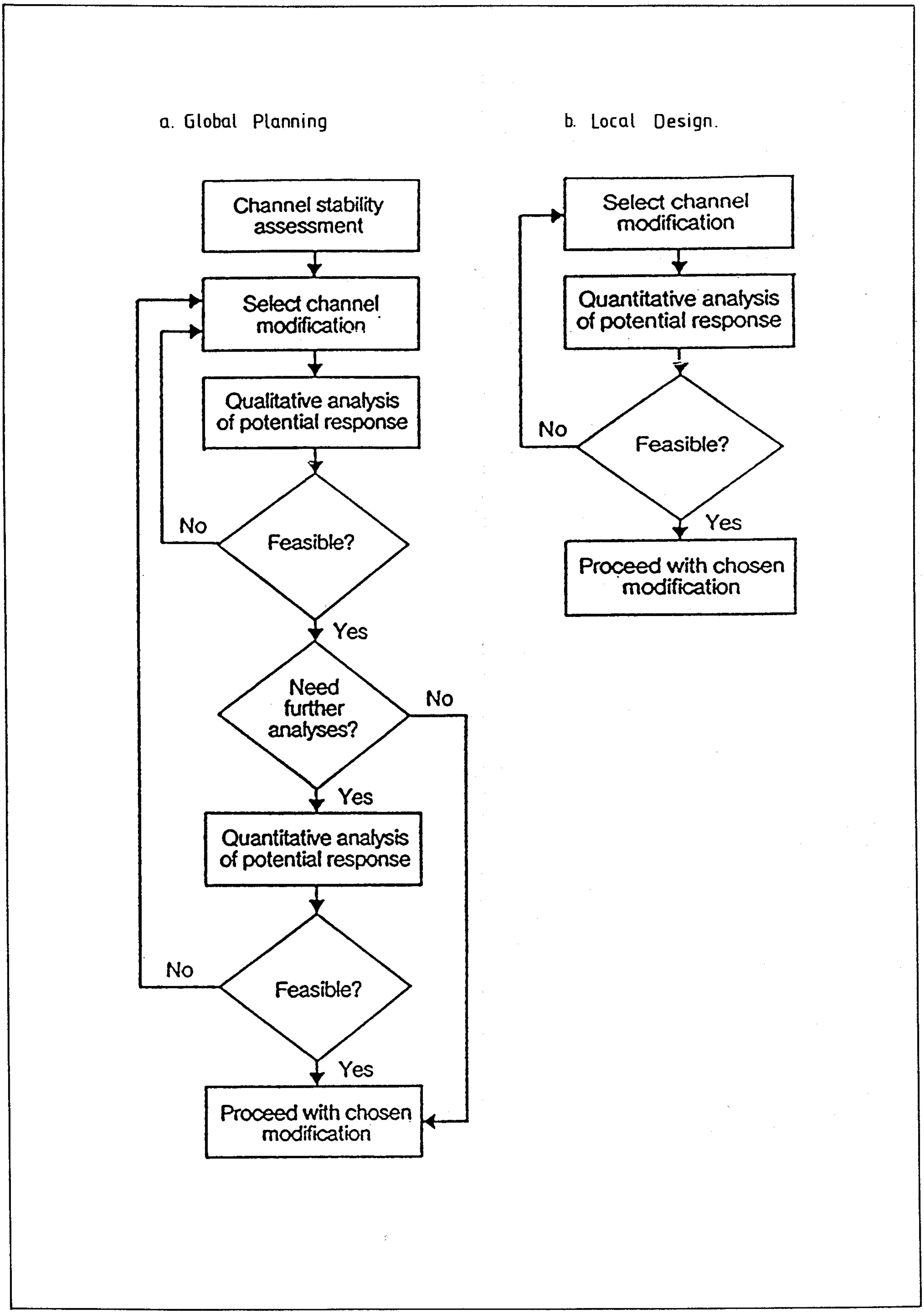


Fig 23 Flowchart for river training project (after Ingram, 1986)

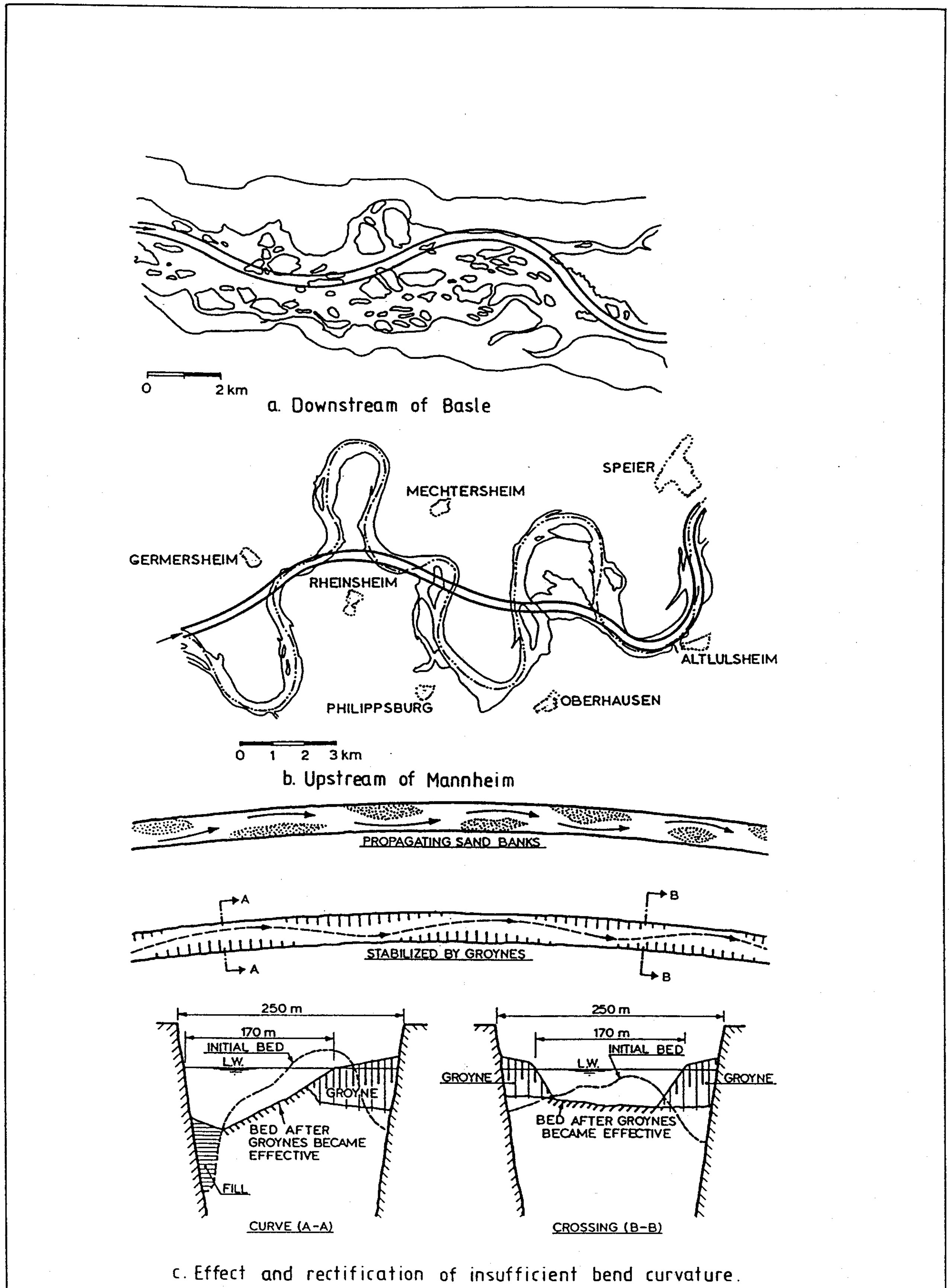
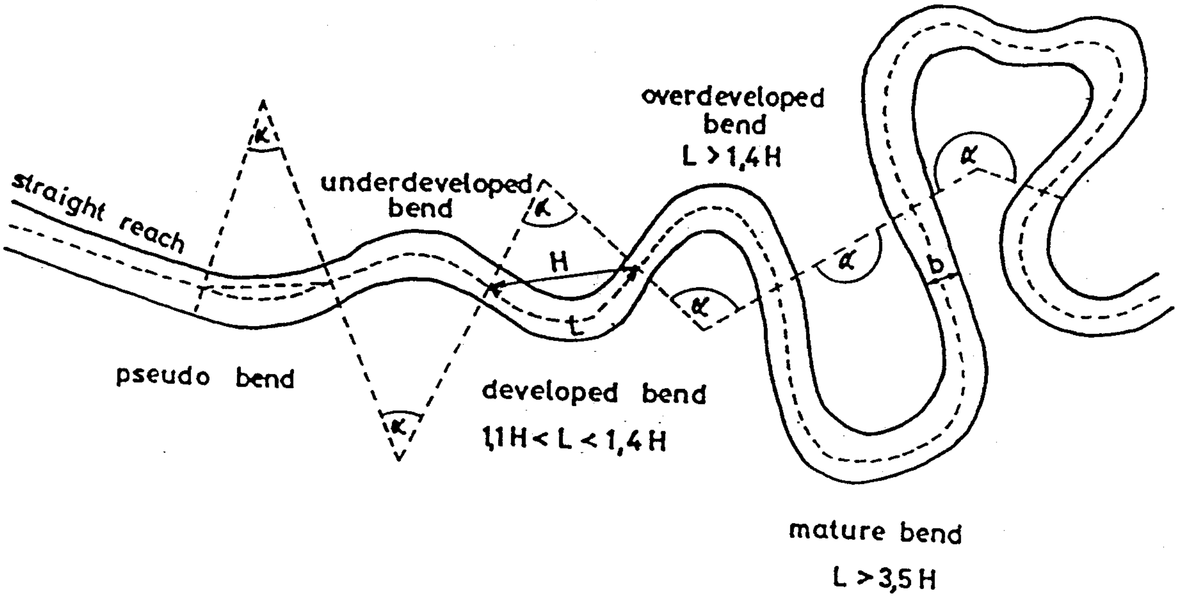
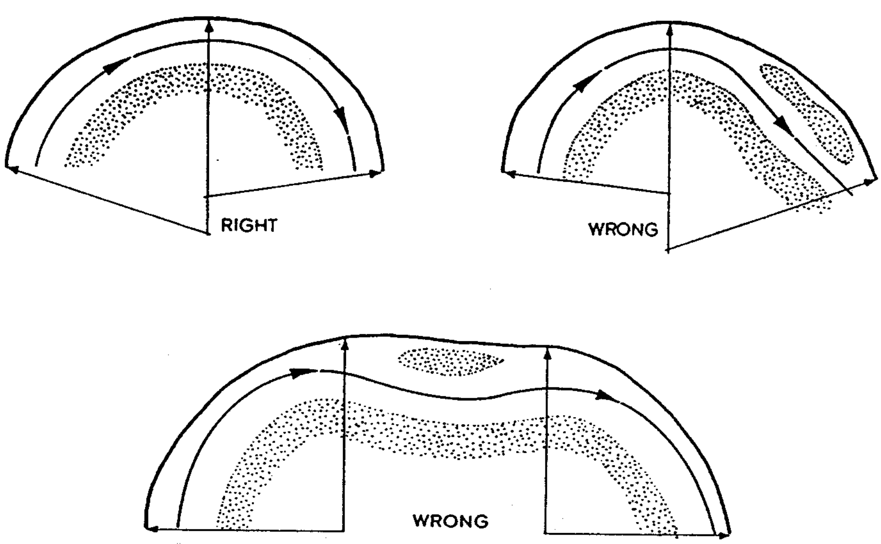


Fig 24 Channelisation of the Rhine (after Jansen et al,1979)



a. Stable and unstable bends (after Bognar, 1987)



b. Ideal bend curvature (Jansen et al, 1979)

Fig 25 Design of stable bends

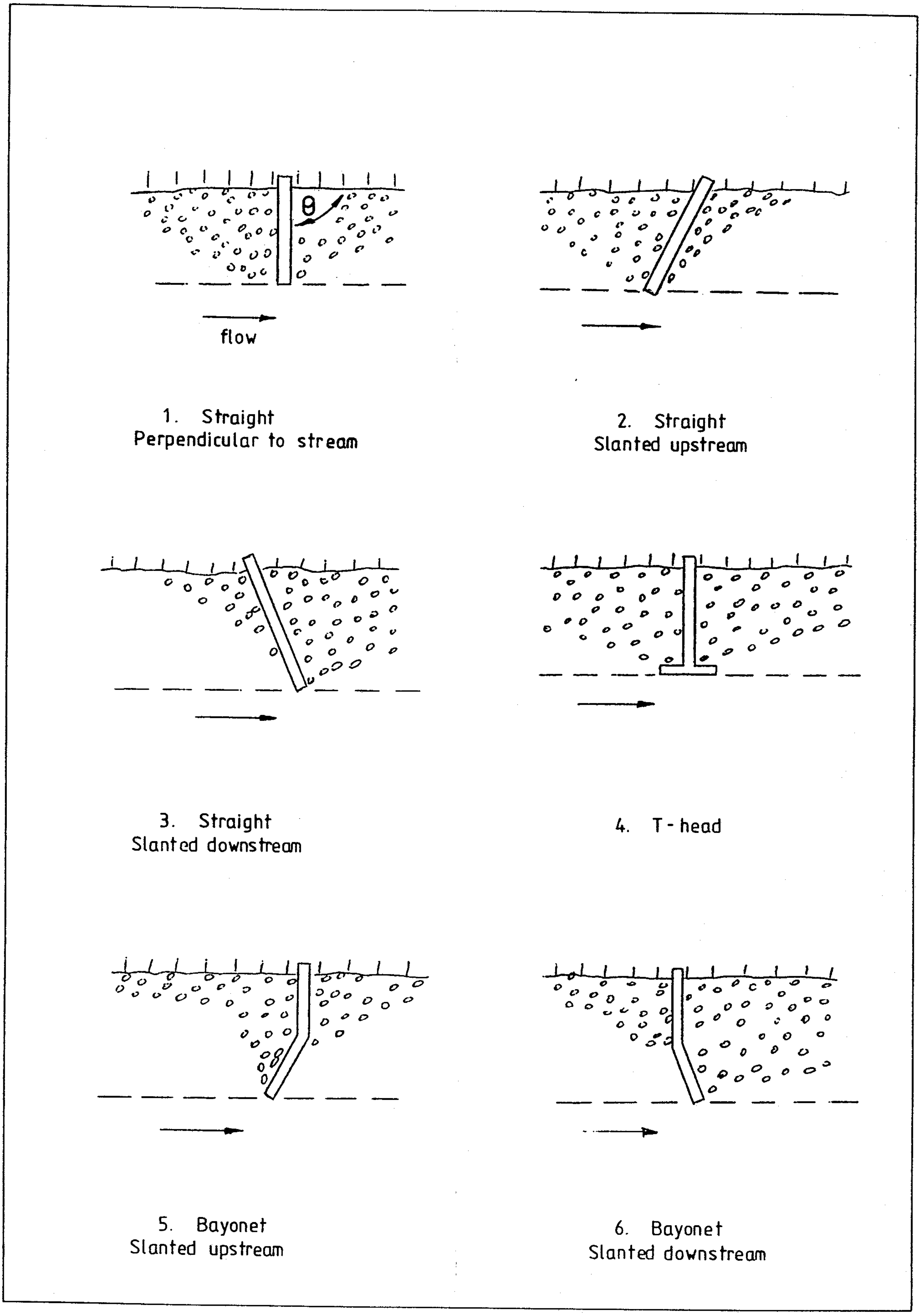


Fig 26 Plan shapes of groynes

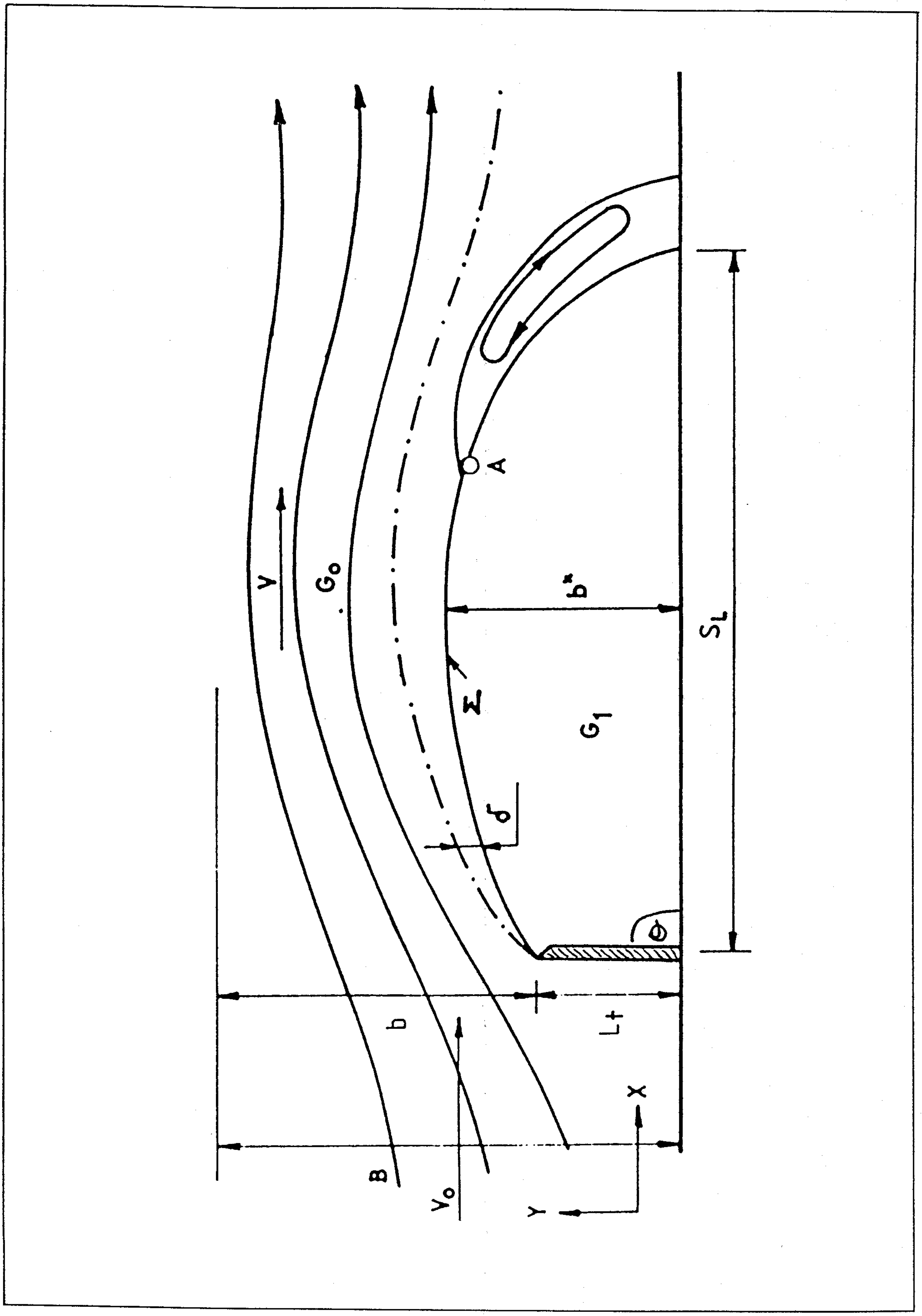


Fig 27 Streamflow around a single groyne

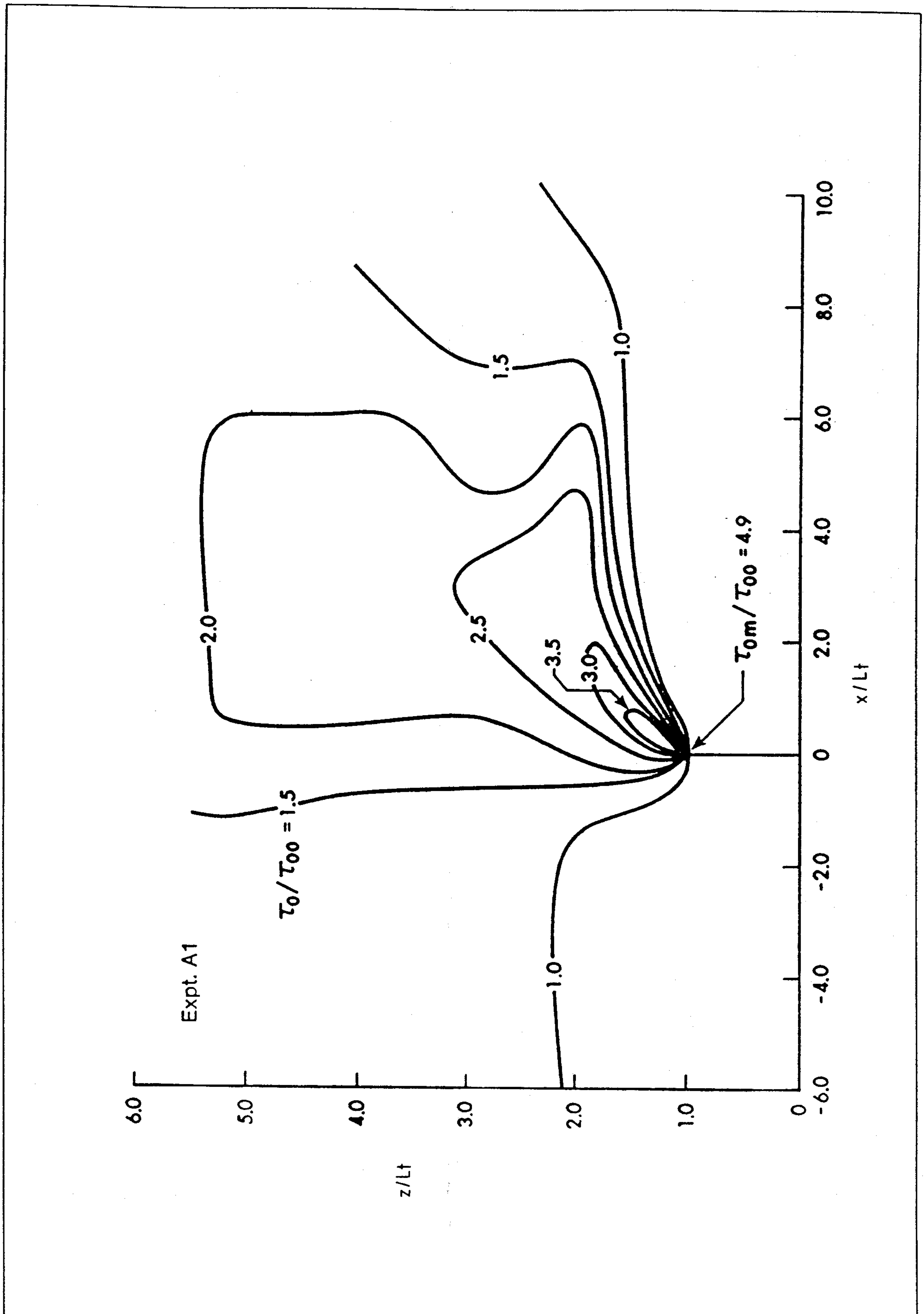


Fig 28 Contours of bed shear stress amplification around a groyne (after Nwachukwu and Rajaratnam, 1980)

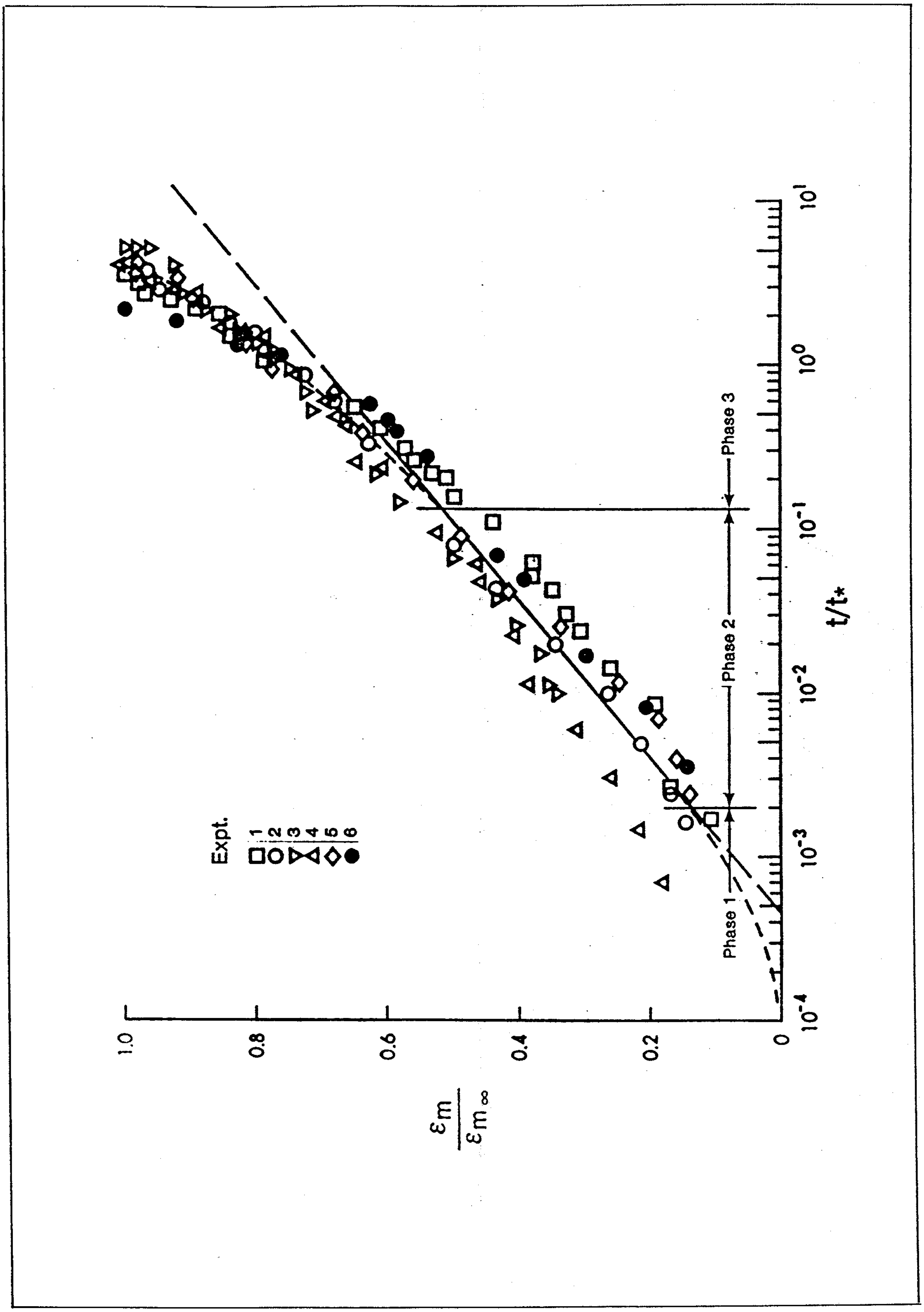
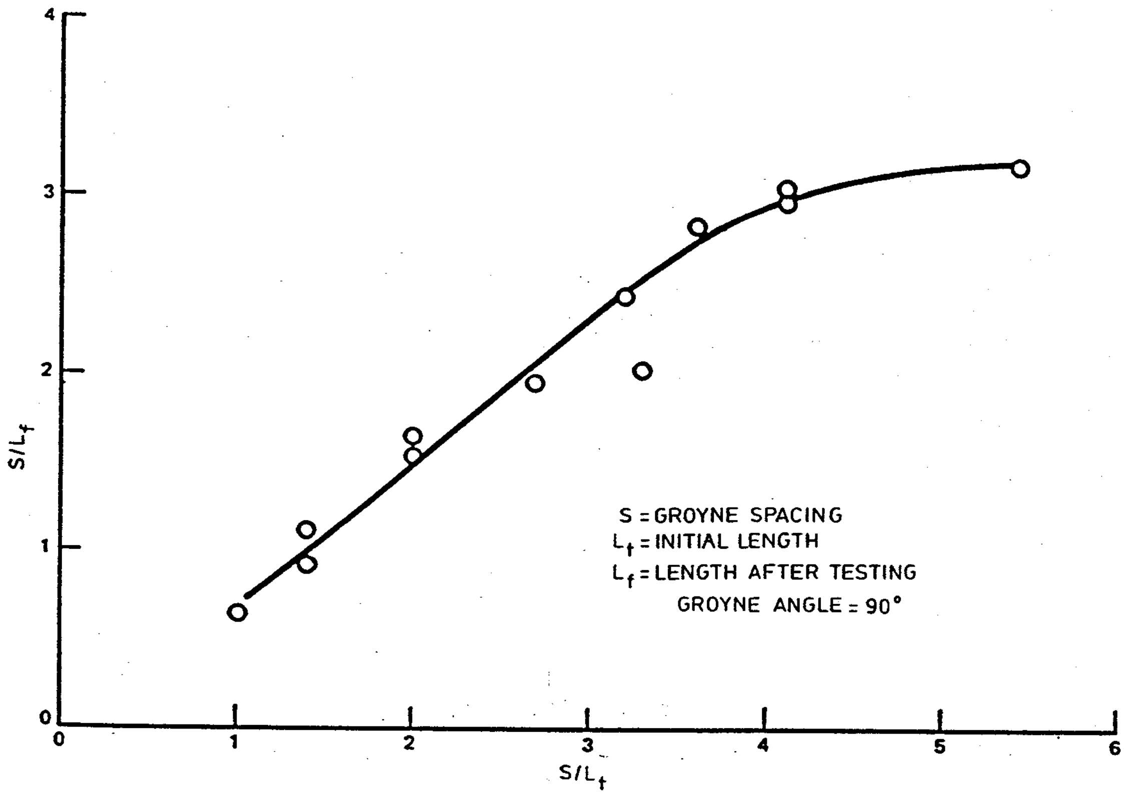
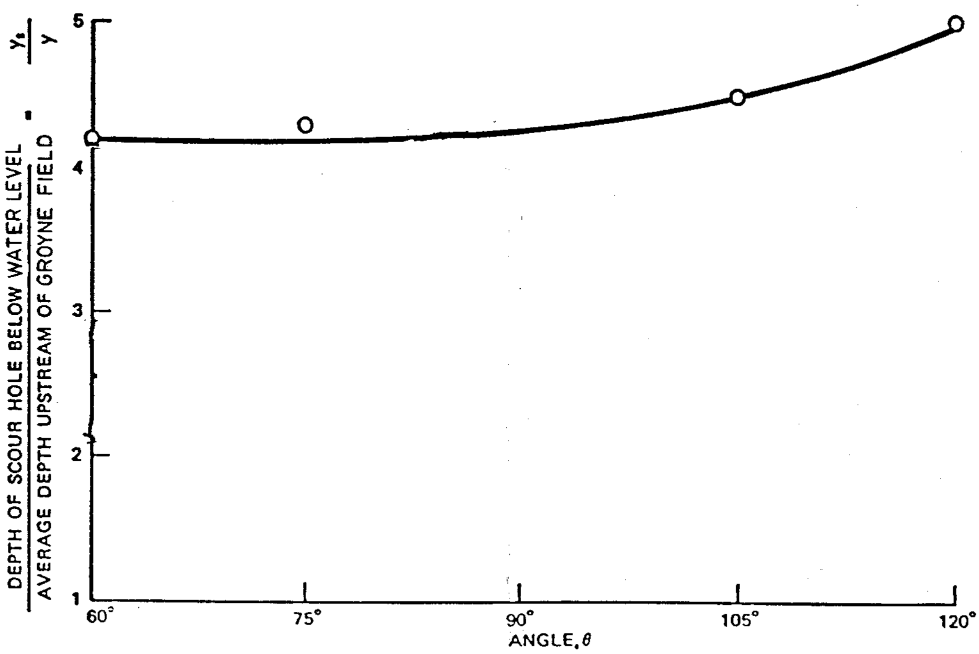


Fig 29 Development of local scour depth with time (after Nwachutwa and Rajaratnam, 1980)



a. Effect of Spacing on length ratio.



b. Effect of groyne angle on scour depths. $S/L_t = 4{\cdot}5$

Fig 30 Copelands experimental results on groyne spacing and orientation.

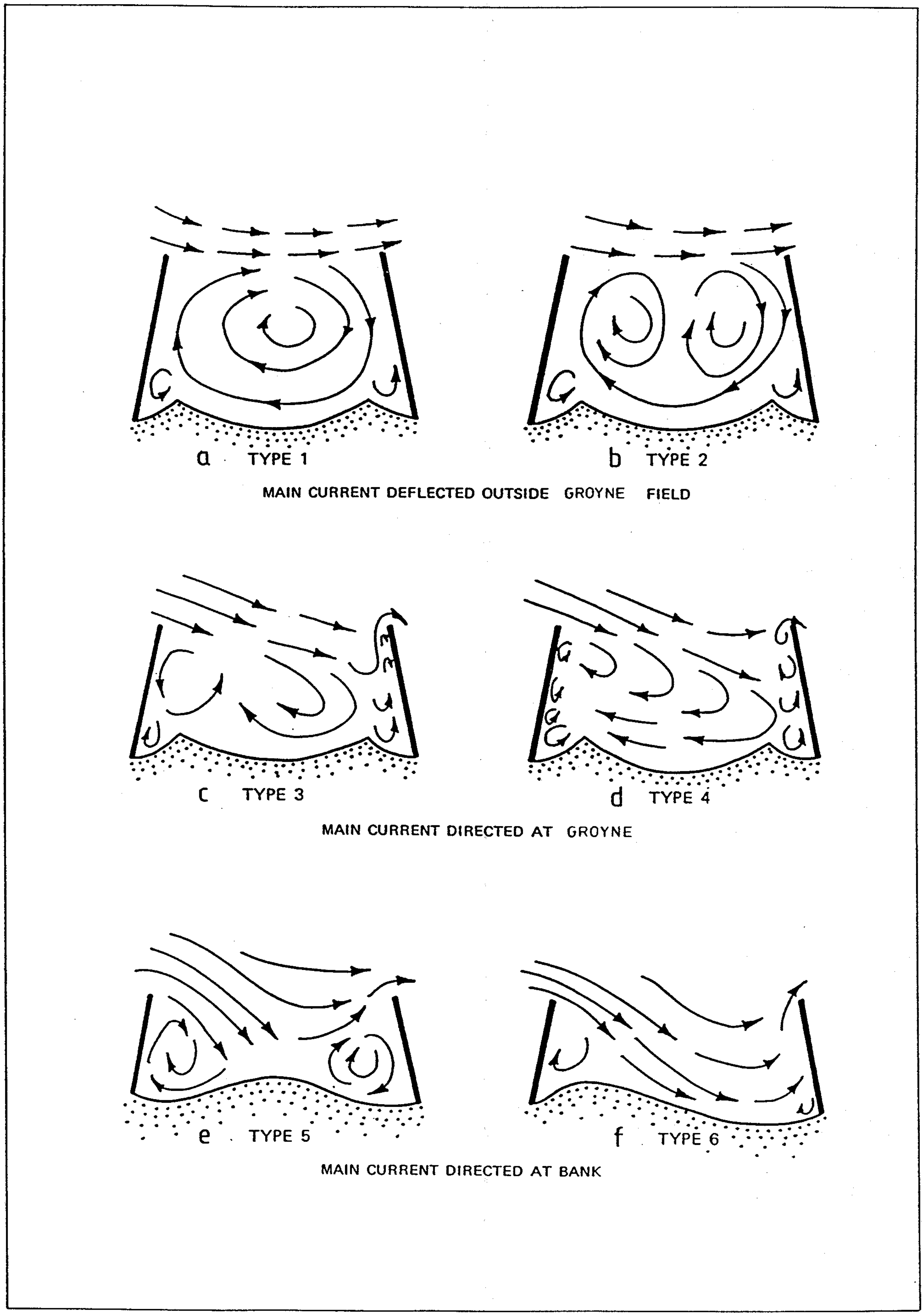


Fig 31 Eddy patterns between groynes (after Copeland 1983)

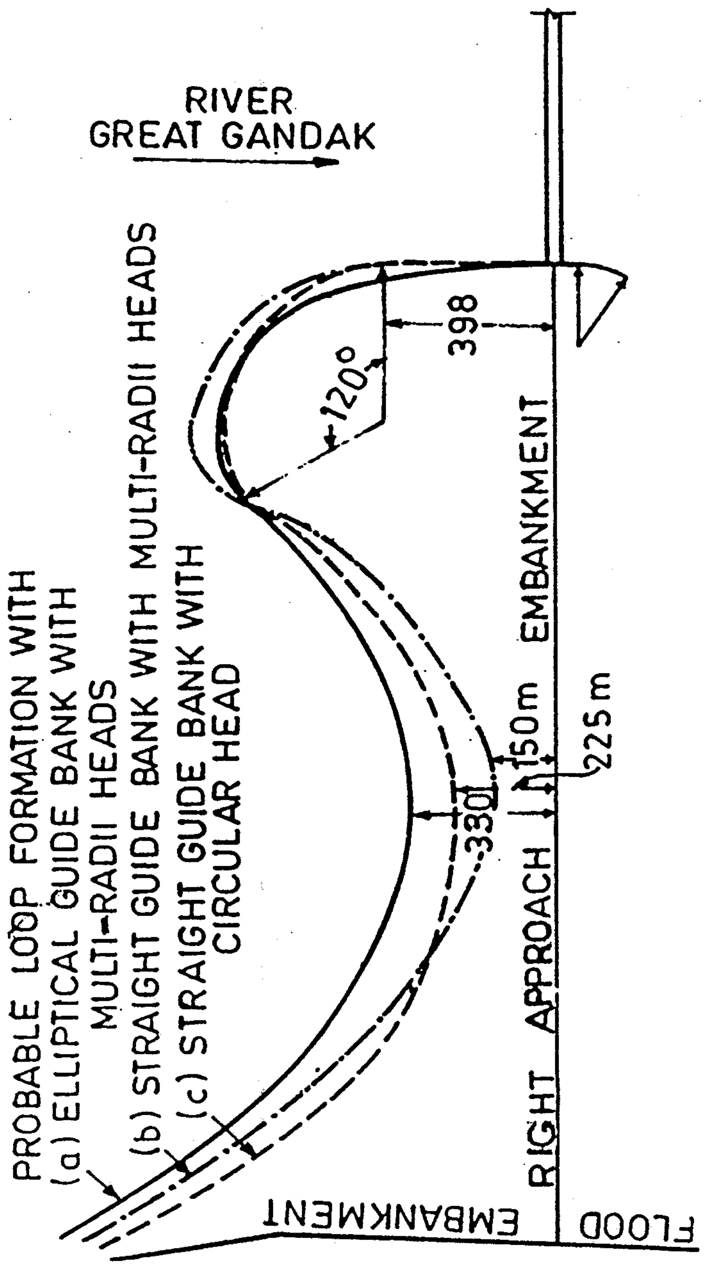


Fig 32 The effect of guide bank shape on meander loop formation (after Garg, 1980)

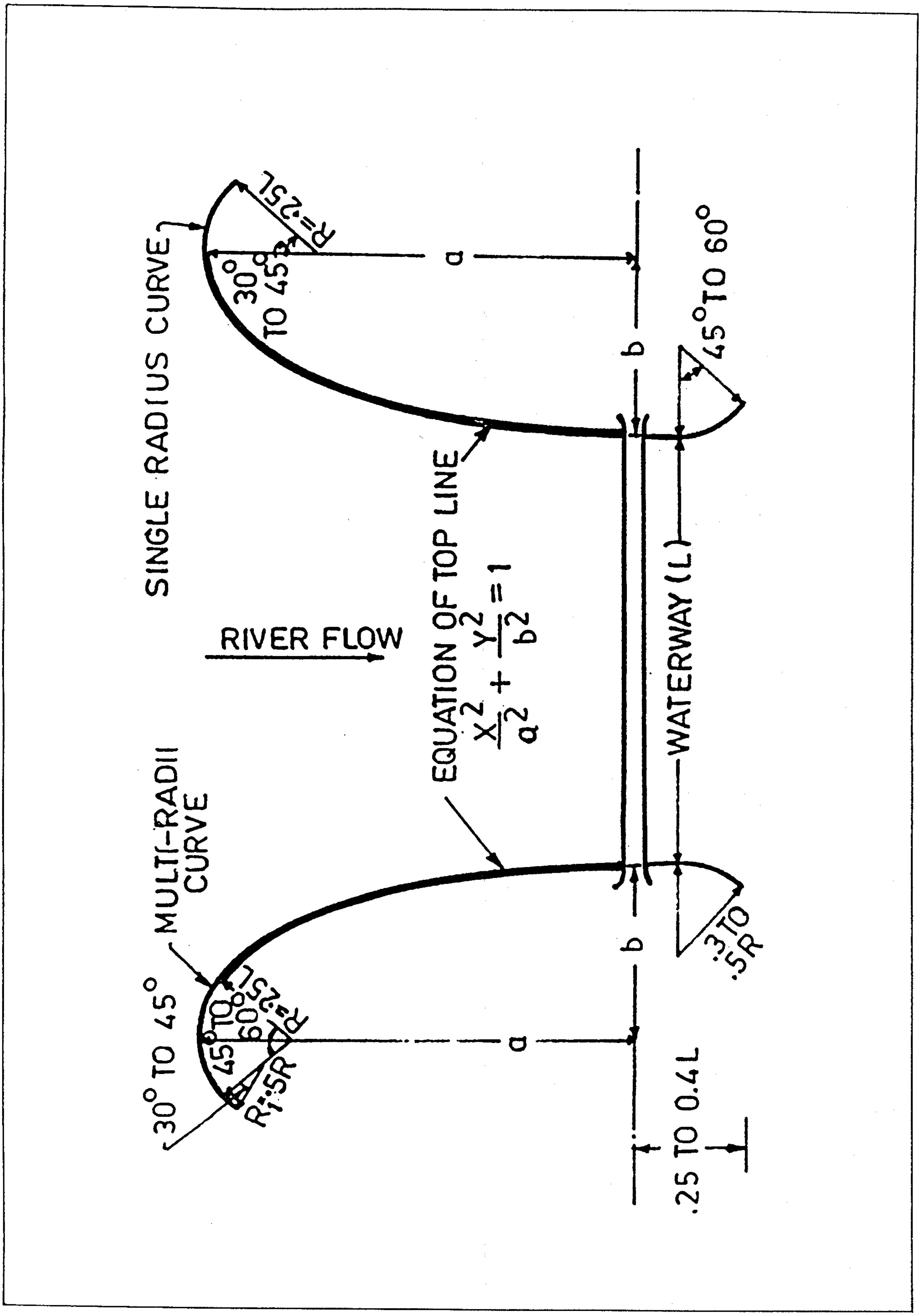


Fig 33 Recommended plan shape of guide banks (after Garg et al, 1980)

APPENDIX

APPENDIX A: General and local scour formulae for groynes

General scour

General scour is the name given to bed lowering across a contracted section due to increased average velocity at the contraction. When training rivers for navigation purposes, this bed lowering is the design objective.

Equations to predict general scour are usually expressed in terms of the contraction ratio, b/B, or its reciprocal. There are two formulae which were developed specifically for channel contraction using groynes :

(1) Anderson and Devenport (1968)

They considered the case of both submerged and unsubmerged groynes. Their analysis is based on shear stress amplification at the constriction. The formula for submerged groyne constrictions was verified by a limited number of flume experiments.

Unsubmerged: $D_2/D_1 = (B/b)^{\Theta}$ where $9/14 \le \Theta \le 6/7$

and where Θ depends upon the shear stress amplification.

Submerged: $D_2/D_1 = (Q_2/Q_1)^{6/7}(B/b)^{\Theta}$ where

$9/14 \le \Theta \le 6/7$

and where Θ depends upon the shear stress amplification and (Q_2/Q_1) is the ratio of flow in the submerged contracted section to the total discharge in the river. This ratio can be taken as:

$$(Q_2/Q_1) = 1/(1+(D_3/D_1)(B/b-1))$$

(2) Wang and Yanapirut (1988)

This equation was derived by fitting coefficients to a relationship derived by dimensional analysis, based on flume experiments with different groyne spacings.

$$(D_2/D_1) = (B/b)^{6/7}(S/L_t)^{-1/7}$$

For a typical range of groyne spacings $2L \le S \le 10L$, the spacing factor is in the range $0.91 \ge (S/L_t) \ge 0.72$.

Both of these formulae are for live bed conditions, ie equilibrium scour is assumed to be when bed shear is the same in the contraction as the main channel. A significant omission is that equilibrium scour depth is assumed to be independent of the bed sediment size.

Local scour

Several researchers have developed equations for predicting the depth of the local scour hole which forms at the nose of a groyne.

(3) Garde et al (1961)

$$(d_s/d) = 4.0\ (B/b)\ \Phi_1\ \Phi_2\ \Phi_3\ \Phi_4\ Frn$$

(4) Liu et al (1961)

$$(d_s/d) = 1 + 1.1\ (L/d)^{0.4} Fr^{1/3} \qquad \text{for } L/d \le 25$$

$$(d_s/d) = 1 + 4.0\ Fr^{1/3} \qquad \text{for } L/d > 25$$

(5) Gill (1972)

$$(d_s/d) = 8.4\ (B/b)^{0.67}\ (D_{50}/d)^{0.25}$$