5. River Training and Flood Control

5.1 Introduction

River training, in its wider aspects, covers all those engineering works which are constructed on a river, so as to guide and confine the flow to the river channel, and to control and regulate the river bed configuration, thus ensuring safe and effective disposal of floods and sediment loads. Stabilizing and training the river along a certain alignment with a suitable waterway is, therefore, the first and foremost aim of river training.

5.2 Objectives of River Training

The term river training implies various measures adopted on a river to stabilize the river channel along a certain alignment with a certain cross section. These measures are required to be adopted because rivers in alluvial plains frequently alter their courses and cause damage to land and properties adjacent to their banks. The main objectives of river training are:

1. To provide a safe passage of flood discharge without overflowing the banks for protection of developed or inhabited area;
2. To prevent the river from changing its course and to avoid outflanking of structures like bridges, weirs, aqueducts, etc.
3. To protect the banks from erosion and improve the alignment by stabilizing the river channel;
4. To deflect the river away from the bank which it might be attacking;
5. To provide minimum depth of flow and a good course for navigation;
6. To transport efficiently the suspended and bed sediment loads.

Depending on the purpose for which a river training program is undertaken, the river training works may be classified as:

1. Training for high water or flood discharge
2. Training for depth or low water training
3. Training for sediment or mean water training

5.2.1 High water training: is undertaken with the purpose of providing safe disposal of maximum floods and thus provide protection against damage due to floods. It is mainly concerned with the most suitable alignment and height of marginal embankments for disposal of floods and may also include other measures of channel improvement for the same purpose. Thus high water training can also be called *Training for Discharge*. Flood protection can be either passive or active.

- **Passive protection:** Such as building on high grounds or on stilts above high water marks; may be even today resorted to as a temporary or isolated protection measure.
- **Active protection:** Allows the continuation of normal human activities in the flood plain during the great part of the flood events.

Complete elimination of flood hazards and ensuing damage for any given region flood plain is practically not feasible, due to the stochastic nature of flood events. It is very seldom, if ever, envisaged from the engineering and economic point of view, since it would lead to unacceptable financial outlays, far outweighing expected damages to property or crops. It follows, therefore, that what characterizes any proposed flood-control project is the extent to which flood damages are expected to be reduced, and not by any means their complete disappearance.

It seems inevitable, therefore, that the problem of damages caused by exceptional flood events will stay with us as long as flood plains remain an important area of human activity.

Engineering methods for flood control and protection are the following:

1. **Stream training and regulation:** Works concerning cross-section, alignment, longitudinal slope & roughness of stream, with the scope of increasing its conveyance capacity.
2. **Reduction of peak discharges:** By means of flood routing through retention reservoirs for temporary storage of floodwaters.
3. **Flood protection by dykes and levees.**
4. **Attenuation of flood waves:** Through diversion to other channels, or to less critical areas.

As flood protection works are actually designed to only reduce the frequency and extent of expected inundation damages, the first question which must be answered when considering any river training project is the determination of the *design discharge.*

Economic benefits from a proposed protection project over the expected useful life span of the works should be equal to or greater than the compound cost of the project. In evaluation of the former, it is generally an accepted practice that only tangible and direct benefits should be drawn into consideration, i.e. those measurable in money equivalent on the one hand, and those accrued directly as the result of the project, on the other hand. Overall cost of the proposed project should include all expenditures required for its completion, operation and maintenance, interest and depreciation.

A simple optimization procedure for a flood control project is schematically shown below.

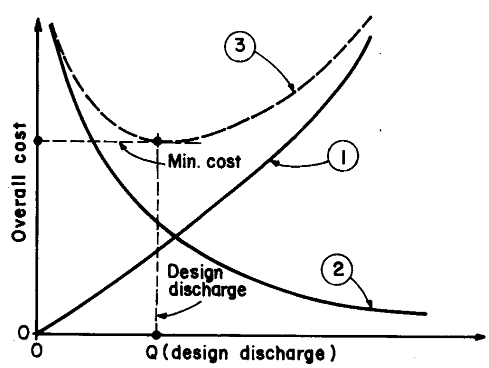

Figure 5.1 Simple optimization procedure

NOTE: *Curve 1: the actual cost of the project*
Curve 2: the remaining average annual flood damages
Curve 3: Sum of curves 1 and 2

The lowest point on curve 3 indicates the minimum annual cost of the project and the corresponding **design discharge**.

5.2.2 Training for Depth or Low Flow: To provide sufficient water depth in navigable channels during low water periods and to maintain aquatic life. It may be accomplished by concentrating and enhancing the flow in the desired channel by closing other channels by the process of bandalling, by contracting the width of the channel with the help of groynes, etc.

5.2.3 Mean Water Training or Training for Sediment: Aims at efficient disposal of suspended and bed load, and thus, to preserve the channel in good shape. The maximum accretion capacity of a river occurs in the vicinity of mean water or dominant discharge. Therefore, the changes in the riverbed are attempted in accordance with that stage of flood flow. The mean water training is the most important type and forms the basis on which the former two are planned. This is so because a river training work adopted to alter the river cross section and alignment must obviously be designed in accordance with that stage of the river at which the maximum movement of sediment takes place during any period under consideration. Although there is maximum activity of the bed of the river at high flow stages but such stages are maintained only for a short duration. On the other hand, there is little movement of sediment at low stages, which persist for a very long duration. In between the two there is a stage at which the combined effect of forces causing sediment movement and the time for which these forces are maintained is maximum. This is therefore the most important stage as it has a considerable influence on the configuration of the river. This stage is somewhere near the mean water.

5.3 Erosion Protection

1. **Groynes or Spurs:** are small jetties *(jetties = structure built out into a water body as a breakwater or landing place for boats, etc*), solid or permeable, constructed of timber, sheet piling, vegetation, and stone rubble, etc. They usually project into the stream perpendicularly to the bank, but sometimes are inclined in the upstream or downstream direction. The main purpose of groynes is to reduce channel width and to remove the damage of scour from the banks.

➢ **Functions of Groynes:**

- Training the river along a desired course by attracting, deflecting, or repelling the flow in the river;
- Creating a slack flow with the objective of silting up the area in the vicinity;
- Protecting the river bank by keeping the flow away from it;
- Contracting a wide river channel, usually for the improvement of depth for navigation.

Groynes can be classified according to methods and materials of construction into:

i. Impermeable groynes

ii. Permeable groynes

i) Impermeable groynes (solid groynes): do not permeate appreciable flow of water through them. They consist of either rockfill, sand and gravel, or soil as available in the river bed, protected on the top and sides by strong stone facing (pitching) or concrete blocks. Slopes vary between 2:1 to 3:1 depending on the material used. Since the head of the groyne is subjected to severe attack by the stream, thicker stone pitching and launching apron is provided. Since the head of the groyne is subjected to severe attack by the stream, it needs protection. Hence at the groyne head a launching apron is provided and also the thickness of the pitching is increased.

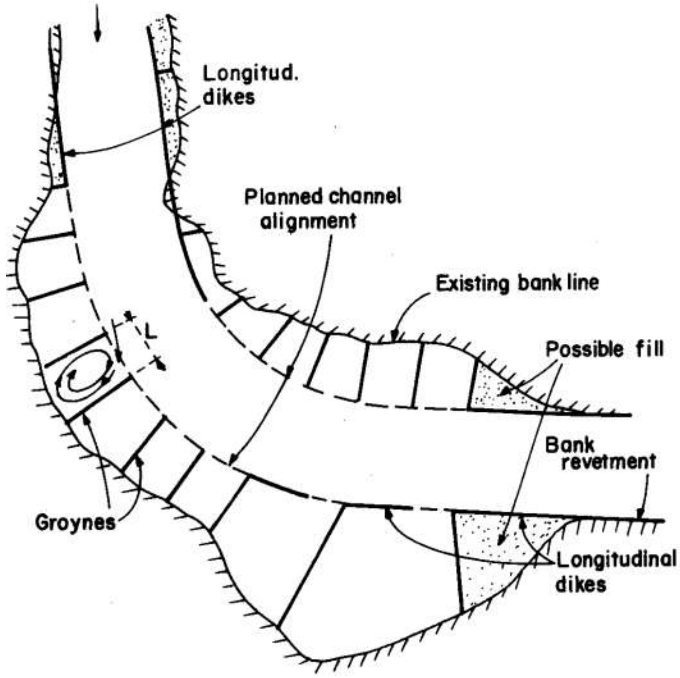

Figure 5.2 River training by groynes

ii) Permeable groynes: permeate restricted flow of water through them. They obstruct the flow and slacken it to cause deposition of sediment carried by the river, hence they are classified as sedimenting groynes and are best suited to rivers carrying considerable suspended sediment. They are made of trees and piles (concrete sheet piles).

iii) Types of groyne alignment

It can be built either perpendicular to the bank line or it may be inclined upstream or downstream.

A groyne pointing upstream has the property of repelling the flow away from it, and scour holes caused by the formation of vertical eddies are developed away from the bank, and near the head of the groyne. Such groynes are called *repelling groynes.* The head of the repelling groyne causes disturbances in the flow at its nose and heavy scour occurs at the nose and slightly downstream of it due to eddy formation. Hence, the head of repelling groyne needs a very strong protection since it is subjected to direct attack of swirling current.

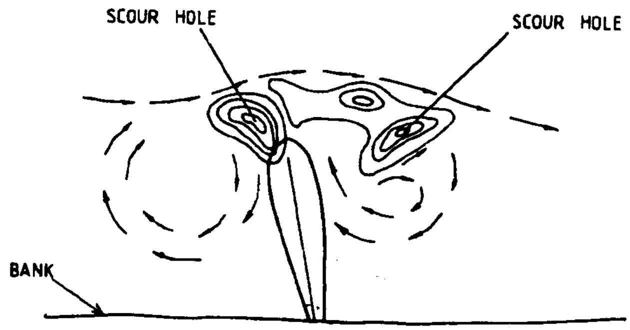

Figure 5.3 Normal groyne (or deflecting groyne)

Groynes pointing downstream have the property of attracting the flow towards them, and are called *attracting groynes.*

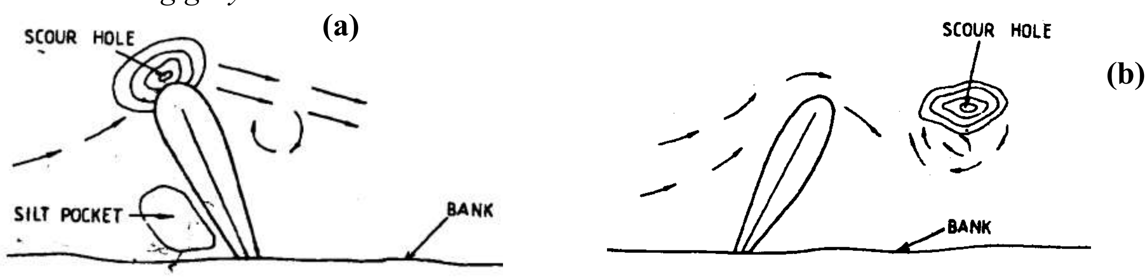

Figure 5.4 (a) Repelling groyne, (b) Attracting groyne

In attracting groynes, the scour holes are developed nearer the bank, as compared to those in repelling groynes. Since such groynes bring the water current as well as scour holes nearer the bank and make it more susceptible to damage, they are generally not used. Further, the main attack of stream on these groynes is on their upstream face and therefore it needs better protection as compared to the downstream face.

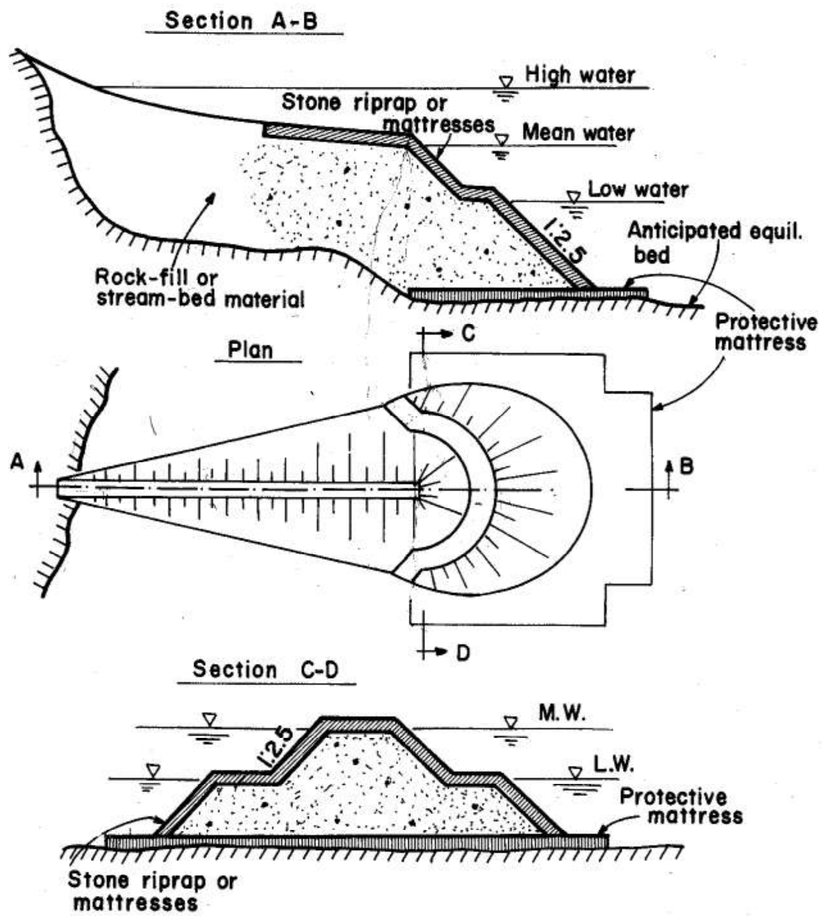

Figure 5.5 Plan and cross section of a groyne (typical groyne structure)

The groynes are, therefore, generally aligned either perpendicular to the bank or pointing upstream. The perpendicular alignment is generally used on convex banks, and the upstream pointing alignment is generally used on concave banks.
When the length of an upstream pointing groyne is small, such that it changes only the direction of flow, without repelling it, it is called *deflecting groyne.*

More extensive studies of local scour around groynes have been carried out by A.M. Gill. The results of his laboratory investigations appear to indicate the following:

(a) Scour depth depends on the depth of flow, and it grows with the increase in depth,
(b) Scour depth depends on the bed material size. For the same value of the ratio τ_c/τ_0: $\tau_0 = \gamma RI$, coarse sand will be scoured deeper than fine sand; on the other hand, for the same absolute value of the shear stress, fine sand will be eroded deeper than the coarse sand (because τ_c for coarse sand is higher than for fine sand).
(c) Bed load movement does not appreciably affect the scour depth. Once the movement of alluvial bed is started, maximum depth of scour tends to remain constant for a given depth of flow.

For design purposes, Gill has proposed an empirical formula (the coefficient has been rounded up):

$$\left[\frac{D}{d}\right]_{max} = 8.4\left[\frac{d_{50}}{d}\right]^{0.25}\left[\frac{B}{b}\right]^{6/7}$$

where D = maximum scour depth below the water surface (m)
d_{50} = median grain size (m)
B = width of the channel before contraction by groynes (m)
b = width of the channel after contraction (m)

The formulas for local scour estimates around embankments are thought to be adequate for groynes also.

For 0< L/d < 25, where L = length of groynes, and d = undisturbed water depth; formula proposed by <u>Liu</u> is used:

$$\left[\frac{S}{d}\right]_{max} = 1.1\left[\frac{L}{d}\right]^{0.4} F_r^{1/3}$$

where S = scour hole depth measured below the mean bed level
F_r = Froude number of undisturbed flow

For L/d > 25, empirical formula obtained from field observations on rock dykes is applied:

$$\left[\frac{S}{d}\right]_{max} = 4F_r^{1/3}$$

Groynes may be constructed either singly or in series, depending upon the need. When constructed in series, they are more effective as they create a pool of almost still water between them, which resist the current and gradually accumulates silt between them, thus forming almost a permanent bank after a certain time. The choice of using them in series arises, if the reach to be protected is long, or if a single groyne is neither strong enough to deflect the current nor quite effective for silt deposition upstream and downstream of itself.

iv) Design considerations

1. Length of groynes: Length depends on the position of the original bank line and the designed normal line of the trained river channel. Too long groynes on easily erodible rivers are susceptible to damage and failure. In such cases, groynes of shorter length may be provided and then they may be extended gradually as silting between them proceeds.

2. Spacing of groynes: Since the length of the bank to be protected by each groyne depends on the length of the groyne, the spacing depends on their length. It is, therefore, taken as a certain proportion of their lengths. Other factors affecting spacing are:

i. *Width of the river*: For rivers of equal flood discharges, a larger ratio of spacing to length of groynes may be used for wide river than for a narrow one.
ii. *Location of groynes:* Large spacing may be used for convex banks than for concave banks (e.g. on concave banks, spacing = length of groynes; on convex banks, spacing = 2 to 2.5 times length of groynes).

iii. *Type of construction (or type of groyne):* Permeable groynes may be spaced farther apart than solid or impermeable ones. Generally, empirical rule of thumb specifies spacing as

- One to two times channel width, or,
- 1 to 5 times the groyne length

According to laboratory tests carried out in the Delft Hydraulic Laboratory, there appears to exist a semi-empirical dependency between the spacing L between the groynes and a theoretically derived parameter. Results of this test seem to indicate that the best flow guiding by groynes is obtained when only one strong eddy is formed between each pair.

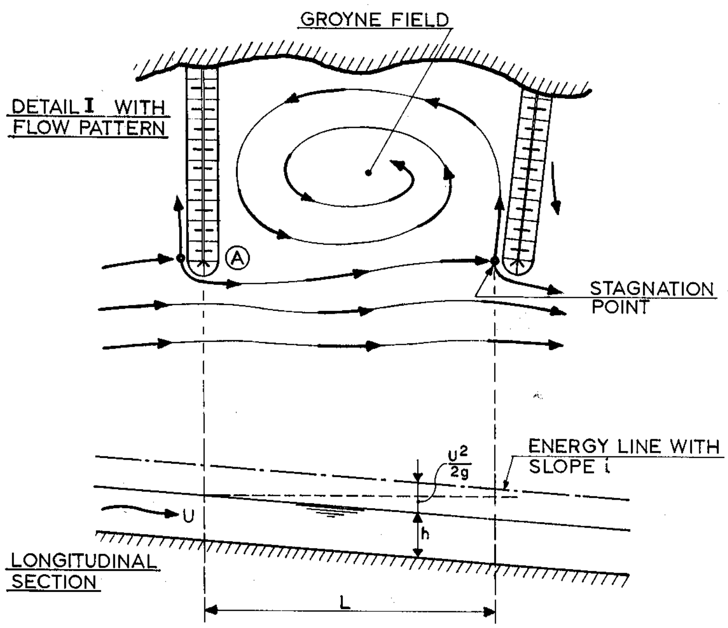

Figure 5.6 Groyne spacing and flow field between groynes

The reasoning follows that the energy required to sustain the backward flow between the groynes can only be available if the energy loss in the stream, IL, is smaller than average velocity head (this is possible only if the level of the water surface in the stagnation point of the downstream groyne is higher than the water level at point A of upstream groyne), $v^2/2g$.

Writing energy equation between the head points of two adjacent groynes and substituting Manning's/ Chezy´s equation to express the mean velocity, it can be easily shown that

$$\mathbf{L = k\frac{d^{4/3}}{2g\, n^2}} \quad \text{; Manning's equation}$$

$$\mathbf{L = k\frac{C^2 d}{2g}} \quad \text{; Chezy's equation}$$

where k is an empirical coefficient

L = spacing (m)

d = mean water depth (m)

n = Manning's roughness coefficient; C = Chezy's coefficient
For strong eddy, coefficient k has been found to be about 0.6 (DHL).

Example:
(a) For d = 4m, n = 0.035
The spacing between groynes should be about

$$L = \frac{0.6x4^{4/3}}{2x9.81x0.035^2} = 159 \approx 160 \text{ m}$$

(a) *For d = 5 m and C = 40 $m^{1/2}/s$*
The spacing between the groynes should be about:

$$L = \frac{0.6x40^2x5}{2x9.81} = 245 \text{ m}$$

In practice, however, the distance L (or spacing) would be taken somewhat less in order to be on the safe side.

2. Stream deflectors (Vanes)

Constructed of wood panels or metal (e.g. floating drums with sheet metal vanes), placed at suitable angle (often almost parallel to the bank) and depth, can be used to either divert an eroding flow from the river bank or, on the other hand, to induce bed erosion and local deepening of the flow. Details of their location are best determined by model studies or experiments in situ.

3. Reduction of Longitudinal Slope by Means of Drop Structures
In the upper reaches of some streams, longitudinal slope is often excessively steep, and hence causes strong erosion. By means of a series of drops along the reach, the slope may be considerably reduced and the erosion-deposition cycle effectively improved.

Reduced slope between the drops is fixed in such a manner that the average shear stress at the design discharge remains below its critical value for the given soil condition. When the regulated slope must be very mild because of soil conditions, and the design discharge is rather high, it is often advantageous to choose a combined solution in which lateral dikes are added.

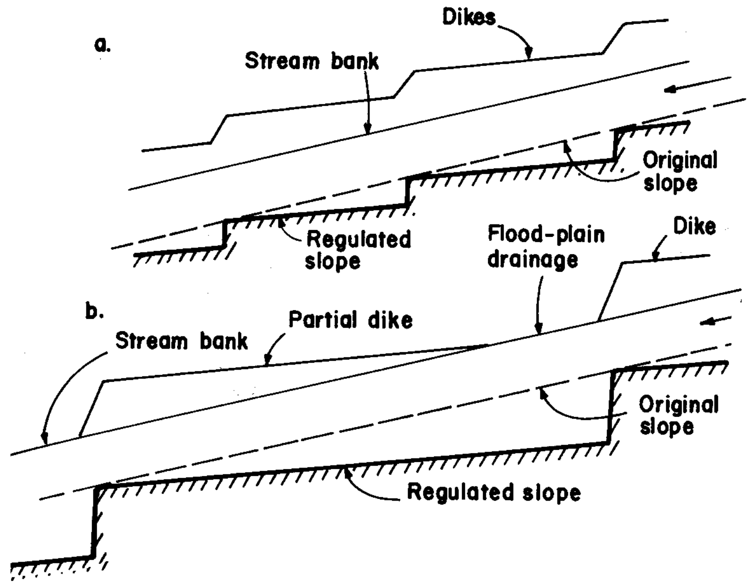

Figure 5.7 Slope reduction by drops

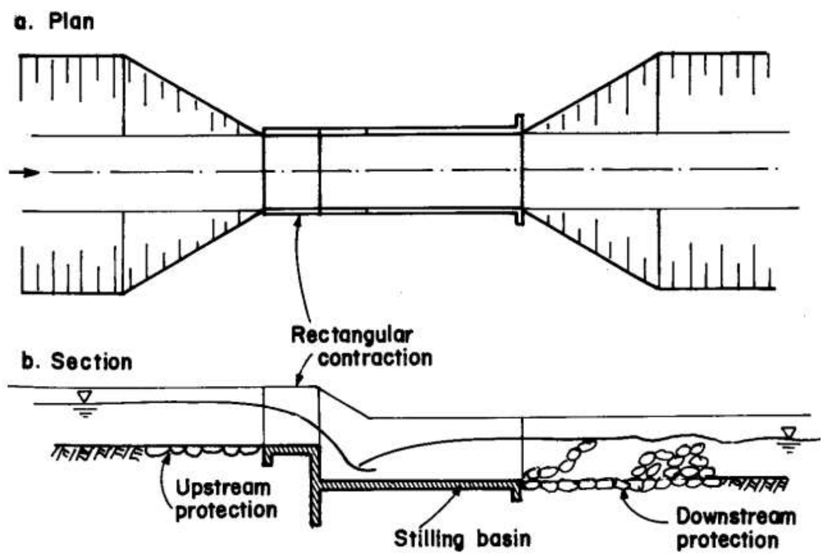

Figure 5.8 A typical drop structure

4. Fixation of the Streambed

It is sometimes preferred to let the bed slope reduction be done essentially by the stream itself. Two such methods are:

a. Small Check-dams

Low check-dams are built across the streambed, at relatively short distances between them. With time, sediment will be deposited between the two dams, and consequently a milder longitudinal slope established. Water level is raised because of the backwater curve upstream of each check-dam, and hence hydraulic gradient is reduced. Bank line is supposed to be high enough to contain the back-up water, since otherwise lateral diking would be required.

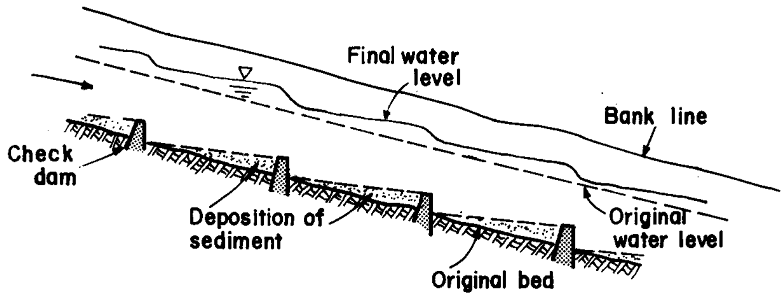

Figure 5.9 Bed fixation by means of check-dams

b. Bottom Sills

Before the erosion of the original bed between the sills takes place, there is no influence of the sills on the flow. Eventually such erosion will go on until a new equilibrium slope is established, milder than the original one. At that stage, the system will form a cascade of small drops. Bottom sills should always be built strong enough to act as low retaining walls after the erosion has taken place.

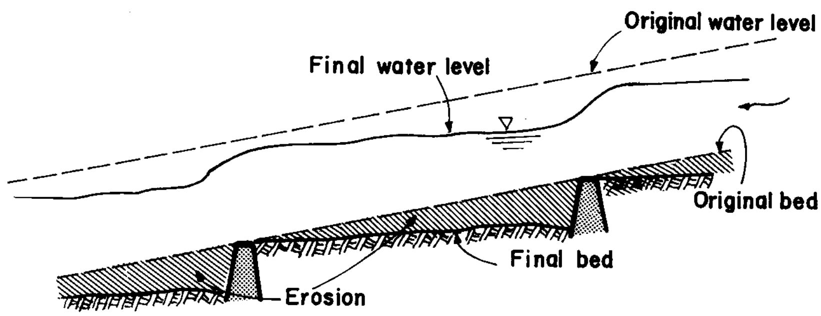

Figure 5.10 Bed fixation by sills

5. Pitched Island

- is an artificially created island in the river bed
- It is protected by some stone pitching on all sides
- A pitched island is created with sand core and boulder lining
- To protect it from scouring, a launching apron is also provided
- The location, size and shape of pitched islands are usually decided on the basis of model studies

- Pitched island serves the following purposes:

1) Creating an oblique approach upstream of weirs, barrages and bridges by training the river bed to be axial

2) Rectifying adverse curvature for effective sediment exclusion

3) Redistributing harmful concentration of flow for relieving attack on marginal bunds, guide banks and river bends, etc
4) Improving the channel for navigation – as barrier increasing depth

6. Stream Bank Protection

Stream banks, even in a regulated channel, are constantly attacked by waves and scoured by the erosive action of the shifting water level. Strong local scour is particularly active along the concave bank of bends. Bank protection may be

- Direct – in which case it is done by a suitable kind of protective revetment,
- Indirect – when the protection against scour is achieved by constructions not directly attached to the banks themselves (e.g., such as groynes)

a. Protection by Plants

On very small streams, the simplest and cheapest protection may be the planting of two to three rows of some sturdy growth. Small seedlings soon develop strong and ramified roots inside the loose soil of the banks, thus conferring to it additional bonding; outside branches and foliage provide shield against the scour.

b. Riprap Revetment

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Riprap is a layer or facing of rock, dumped or hand-placed to prevent erosion, scour, or sloughing of a structure or embankment. Materials other than rock are also referred to as riprap; for example, rubble, broken concrete slabs, and preformed concrete shapes (slabs, blocks, rectangular prisms, etc.).

In this context, riprap is defined as:
"A flexible channel or bank lining or facing consisting of a well graded mixture of rock, broken concrete, or other material, usually dumped or hand-placed, which provides protection from erosion."

As described above, riprap is a flexible revetment. Flexibility of the riprap mass is due to individual particles acting independently within the mass.

Rock riprap is the most widely used and most desirable type of revetment. It is compatible with most environmental settings. The term "riprap" is most often used to refer to rock riprap. Rock riprap is further subdivided by placement method into dumped riprap, hand-placed riprap, and plated or keyed riprap.

Riprap is composed of three sections: the armor or stone layer, the filter layer, and the toe protection. Typical armor is composed of rough, angular rock. The second component, the underlying filter layer, supports the stone against settlement, allows groundwater to drain through the structure, and prevents the soil beneath from being washed through the armor layer by waves or groundwater seepage.

Dumped riprap is graded stone dumped on a prepared slope in such a manner that segregation will not take place. Dumped riprap forms a layer of loose stone; individual stones can independently adjust to shifts in or movement of the base material.

Advantages associated with the use of dumped rock riprap include:

- The riprap blanket is flexible and is not impaired or weakened by minor movement of the bank caused by settlement or other minor adjustments.
- Local damage or loss can be repaired by placement of more rock.
- Construction is not complicated.
- When exposed to fresh water, vegetation will often grow through the rocks, adding esthetic and structural value to the bank material and restoring natural roughness.
- Riprap is recoverable and may be stockpiled for future use.

Hand-placed riprap is stone laid carefully by hand or by derrick following a definite pattern, with the voids between the larger stones filled with smaller stones and the surface kept relatively even. The need for interlocking stone in a hand-placed revetment requires that the stone be relatively uniform in size and shape (square or rectangular).

Advantages associated with the use of hand-placed riprap include:

- The even interlocking surface produces a neat appearance and reduces flow turbulence at the water revetment interface.
- The support provided by the interlocking of individual stones permits the use of hand-placed riprap revetments on steeper bank slopes than is possible with the same size loose stone riprap.
- With hand-placed riprap, the blanket thickness can usually be reduced to 150 to 300 mm less than a loose riprap blanket, resulting in the use of less stone.

Disadvantages associated with hand-placed riprap include:

- Installation is very labor-intensive, resulting in high costs.
- The interlocking of individual rocks in hand-placed revetments results in a less flexible revetment; as mentioned above, a small shift in the base material of the bank can cause failure of large segments of the revetment.
- By their nature, hand-placed rock riprap revetments are more expensive to repair than are loose rock revetments.

Plated or keyed riprap is similar to hand-placed riprap in appearance and behavior, but different in placement method. Plated riprap is placed on the bank with a skip and then tamped into place using a steel plate, thus forming a regular, well-organized surface. Experience indicates that during the plating operation, the larger stones are fractured, producing smaller rock sizes to fill voids in the riprap blanket.

Advantages and disadvantages associated with the use of plated riprap are similar to those listed above for hand-placed riprap.

Extent of Protection: Extent of protection refers to the longitudinal and vertical extent of protection required to adequately protect the channel bank. The longitudinal extent of protection required for a particular bank protection scheme is highly dependent on local site conditions.

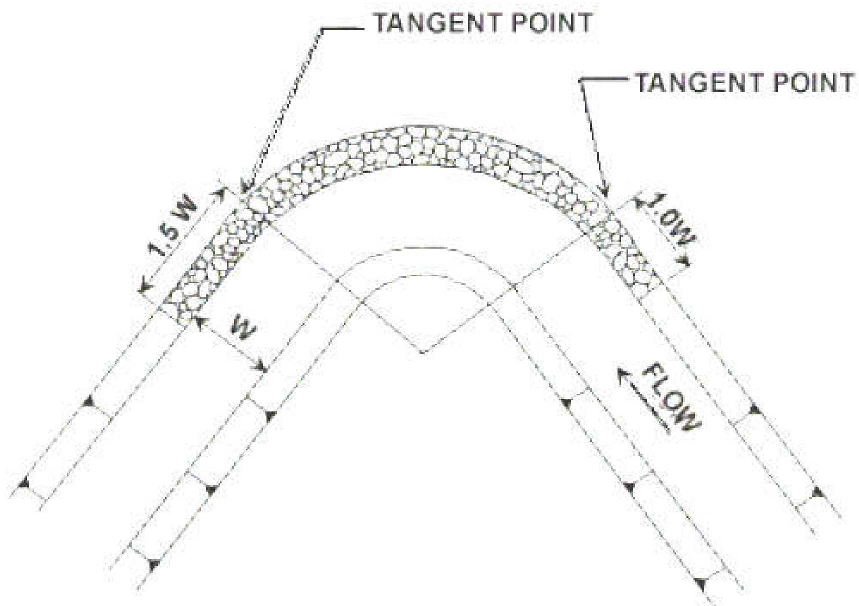


Figure 5.11 Longitudinal Extent of Revetment Protection

In general, the revetment should be continuous for a distance greater than the length that is impacted by channel-flow forces severe enough to cause dislodging and/or transport of bank material.

One criterion for establishing the longitudinal limits of protection required is illustrated in Figure 4.19. As illustrated, the minimum distances recommended for bank protection are an upstream distance of 1.0 channel width and a downstream distance of 1.5 channel widths from corresponding reference lines (see Figure 4.19). All reference lines pass through tangents to the bend at the bend entrance or exit.

This criterion is based on analysis of flow conditions in symmetric channel bends under ideal laboratory conditions.

Toe Depth: The undermining of revetment toe protection has been identified as one of the primary mechanisms of riprap revetment failure. In the design of bank protection, estimates of the depth of scour are needed so that the protective layer is placed sufficiently low in the streambed to prevent undermining. The ultimate depth of scour must consider channel degradation as well as natural scour and fill processes.

The relationships presented in the following equation can be used to estimate the probable maximum depth of scour due to natural scour and fill phenomenon in straight channels, and in channels having mild bends.

In application, the depth of scour, d_s, determined from the equation should be measured from the lowest elevation in the cross section. It is assumed that the low point in the cross section may eventually move adjacent to the riprap.

$$d_s = 3.66 \text{ m for } D_{50} < 0.0015 \text{ m},$$
$$d_s = 1.74\, D_{50}^{-0.11} \text{ for } D_{50} > 0.0015 \text{ m},$$

Where d_s = estimated probable maximum depth of scour (m)
D_{50} = median diameter of bed material (m)

The depth of scour predicted by this Equation must be added to the magnitude of predicted degradation and local scour (if any) to arrive at the total required toe depth.

Construction

Bank slope: A primary consideration in the design of stable riprap bank protection schemes is the slope of the channel bank. For riprap installations, the maximum recommended face slope is 1V: 2H.

Bank Preparation: The bank should be prepared by first clearing all trees and debris from the bank, and grading the bank surface to the desired slope. In general, the graded surface should not deviate from the specified slope line by more than 150 mm.

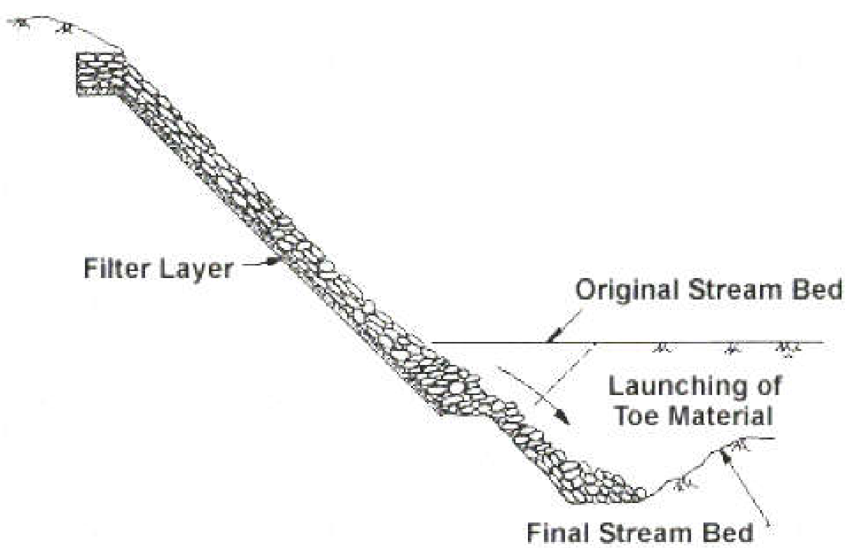


Figure 5.12a. Typical Riprap Installation: End View (bank protection only)

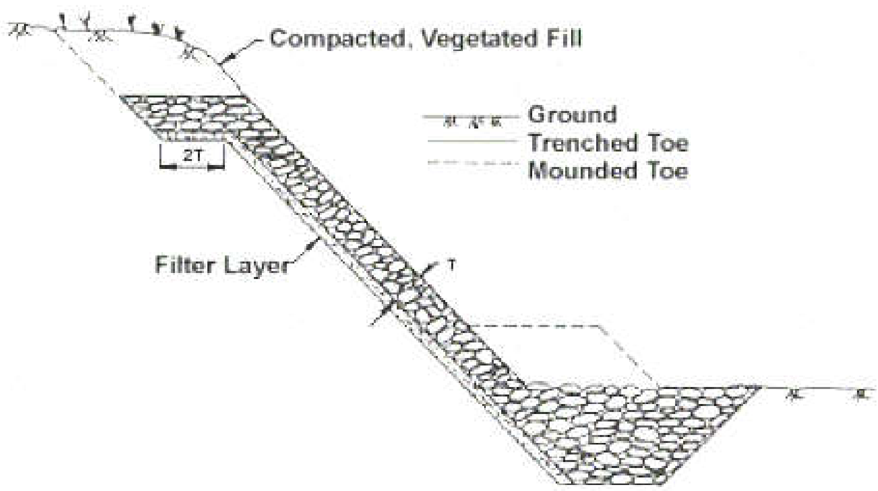


Figure 5.12b. Launching of Riprap Toe Material

Design Guidelines for Rock Riprap

Rock Size: The stability of a particular riprap particle is a function of its size, expressed either in terms of its weight or equivalent diameter.

A riprap design relationship that is based on tractive force theory yet has velocity as its primary design parameter is presented below. The design relationship is based on the assumption of uniform, gradually varying flow. The following form can be used to calculate D_{50}:

$$D_{50} = 0.00594\, V_a^3 / (d_{avg}^{0.5}\, K_1^{1.5})$$

where:
D_{50} = the median riprap particle size (m); C = correction factor (described below); V_a = the average velocity in the main channel (m/s); d_{avg} = the average flow depth in the main flow channel (m); and d_{avg} = the average flow depth in the main flow channel (m); and K_1 is defined as:

$$K_1 = [1 - (\sin^2\theta / \sin^2\phi)]^{0.5}$$

where:
θ= the bank angle with the horizontal; and φ= the riprap material's angle of repose.

The average flow depth and velocity used are main channel values. The main channel is defined as the area between the channel banks.

The above equation is based on a rock riprap specific gravity of 2.65, and a stability factor of 1.2. The following equations present correction factors for other specific gravities and stability factors.

The following form can be used to calculate D_{50}:

$$C_{sg} = 2.12/(S_s - 1)^{1.5}$$

Where S_s = the specific gravity of the rock riprap.

$$C_{sf} = (SF/1.2)^{1.5}$$

Where SF = the stability factor to be applied.

The two correction factors computed are multiplied together to form a single correction factor C. This correction factor, C, is then multiplied by the riprap size computed earlier to arrive at a stable riprap size.

The stability factor is defined as the ratio of the riprap material's critical shear stress and the average tractive force exerted by the flow field. As long as the stability factor is greater than 1, the critical shear stress of the material is greater than the flow induced tractive stress, the riprap is considered to be stable. As mentioned above, a stability factor of 1.2 was used in the development of the equation.

Channel Bends: At channel bends modifications to the stability factor are recommended based on the ratio of curve radius to channel width (R/W) as indicated in the following:

R/W	Stability Factor
> 30	1.2
30 > R/W > 10	1.3 - 1.6
< 10	1.7

Rock Gradation: The gradation of stones in riprap revetment affects the riprap's resistance to erosion. The stone should be reasonably well graded throughout the riprap layer thickness.

Layer Thickness: The following criteria apply to the riprap layer thickness:

- It should not be less than the spherical diameter of the D_{100} (W_{100}) stone, or less than 1.5 times the spherical diameter of the D_{50} (W_{50}) stone, whichever results in the greater thickness.
- It should not be less than 300 mm for practical placement.
- The thickness determined by either 1 or 2 should be increased by 50 percent when the riprap is placed underwater to provide for uncertainties associated with this type of placement.
- An increase in thickness of 150-300 mm, accompanied by an appropriate increase in stone sizes, should be provided where riprap revetment will be subject to attack by floating debris or ice, or by waves from boat wakes, wind, or bedforms.

Wire-Enclosed Rock or Gabions

Wire-enclosed rock, or gabion, revetments consist of rectangular wire mesh baskets filled with rock. These revetments are formed by filling pre-assembled wire baskets with rock, and anchoring to the channel bottom or bank. Wire-enclosed rock revetments are generally of two types distinguished by shape:

1) **Rock and Wire-Mattresses**, In mattress designs, the individual wire mesh units are laid end to end and side to side to form a mattress layer on the channel bed or bank. The gabion baskets comprising the mattress generally have a depth dimension, which is much smaller than their width or length.
2) **Block Gabions**, On the other hand, are more equidimensional, having depths that are approximately the same as their widths, and of the same order of magnitude as their lengths. They are typically rectangular or trapezoidal in shape. Block gabion revetments are formed by stacking the individual gabion blocks in a stepped fashion.

As revetments, wire-enclosed rock has limited flexibility. They will flex with bank surface subsidence; however, if excessive subsidence occurs, the baskets will span the void until the stresses in rock-filled baskets exceed the tensile strength of the wire strands. At this point the baskets will fail.

The conditions under which wire-enclosed rock is applicable are similar to those of other revetments. However, their economic use is limited to locations where the only rock available economically is too small for use as rock riprap slope protection.

The primary advantages of wire-enclosed rock revetments include:
- Their ability to span minor pockets of bank subsidence without failure.
- The ability to use smaller, lower quality, and less dense, rock in the baskets.

The primary disadvantages of wire-enclosed rock revetments include:
- Susceptibility of the wire baskets to corrosion and abrasion damage.
- High labor costs associated with fabricating and filling the wire baskets.
- More difficult and expensive repair than standard rock protection.
- Less flexibility than standard rock protection.

Besides its use as a general bank revetment, wire-enclosed rock in the form of either mattresses or blocks is also used as bank toe protection. In some instances the wire-enclosed rock is used alone for protection of the bank also. In other cases, the wire-enclosed rock is used as toe protection along with some other bank revetment.

The most common failure mechanism of wire basket revetments has been observed to be failure of the wire baskets.

Wire-enclosed rock revetments are classified by geometry as mattress or block type revetments. **Rock and wire mattress revetments** consist of flat wire baskets. The individual mattress sections are laid end-to-end and side-to-side on a prepared channel bed or bank to form a continuous mattress. The individual basket units are attached to each other and anchored to the base material.

Block gabion revetments consist of rectangular wire baskets, which are stacked in a stepped-back fashion to form the revetment surface. Gabion baskets are best used as bank protection where the bank is too steep for conventional rock riprap revetments. Gabion baskets can be stacked to form almost vertical banks (looking much like retaining walls) making them useful in areas where the banks cannot economically be graded to the stable slope required for other riprap types.

Wire-enclosed rock (gabion) revetments consist of rectangular wire mesh baskets filled with rock. The most common types of wire-enclosed revetments are mattresses and stacked blocks. The wire cages, which make up the mattresses and gabions, are available from commercial manufacturers. If desired, the wire baskets can also be fabricated from available wire fencing materials.

Mattresses: Rock and wire mattress revetments consist of flat wire baskets or units filled with rock that are laid end to end and side to side on a prepared channel bed and/or bank. The individual mattress units are wired together to form a continuous revetment mattress.

- **Bank and Foundation Preparation:** Channel banks should be graded to a uniform slope.
- **Mattress Unit Size and Configuration**: Individual mattress units should be a size that is easily handled on site. Commercially available gabion units come in standard sizes. The mattress should be divided into compartments so that failure of one section of the mattress will not cause loss of the entire mattress. Compartmentalization also adds to the structural integrity of individual gabion units. It is recommended that diaphragms be installed at a nominal 0.91 m spacing within each of the gabion units to provide the recommended compartmentalization. On steep slopes (greater than 3H: 1V), and in environments subject to high stresses (in areas prone to high flow velocities, debris flows, ice flows, etc.), diaphragms should be spaced at minimum intervals of 0.61 m to prevent movement of the stone inside the basket.

- **Stone Size:** The maximum size of stone should not exceed the thickness of individual mattress units. The stone should be well graded within the sizes available, and 70 percent of the stone, by weight, should be slightly larger than the wire-mesh opening.

- **Stone Quality**: The stone should meet the quality requirements as specified for dumped-rock riprap.

- **Basket Fabrication:** Commercially fabricated basket units are formed from galvanized steel wire mesh of triple twist hexagonal weave. Wire mattress units may also be fabricated from available fencing materials.

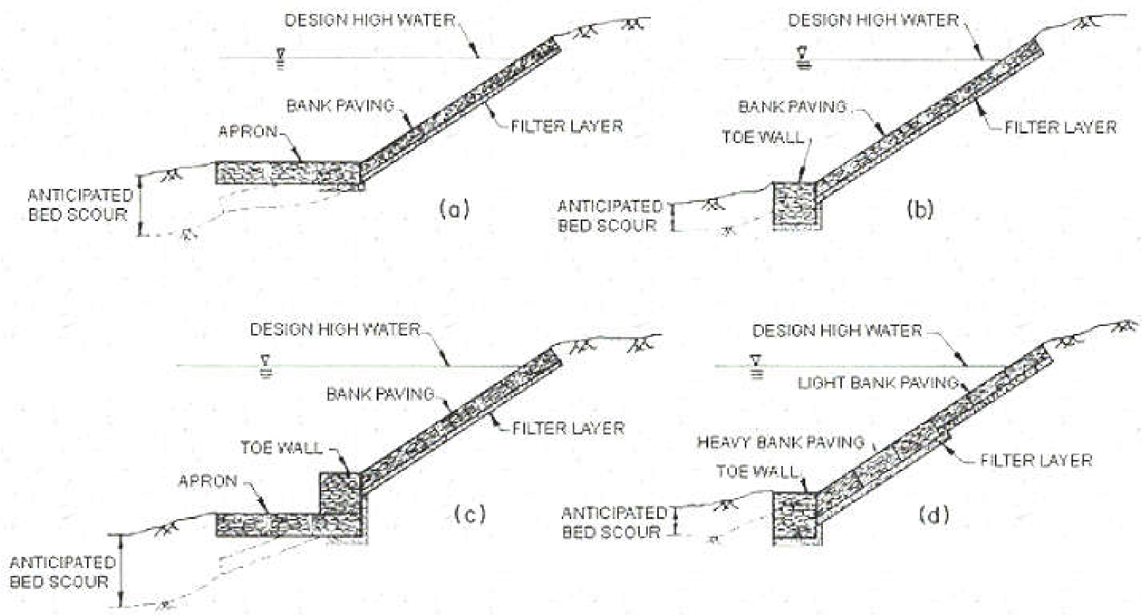


Figure 5.13 Rock and Wire Mattress Configurations: (a) Mattress with Toe Apron; (b) Mattress with Toe Wall; (c) Mattress with Toe Wall; and (d) Mattress of Variable Thickness

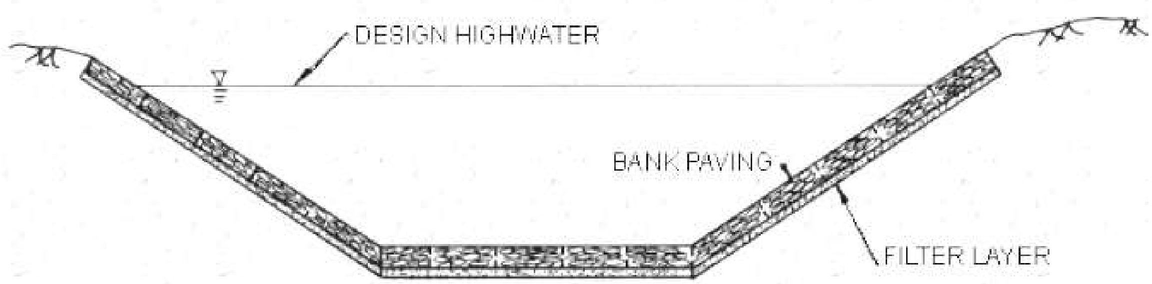


Figure 5.14 Rock and Wire Mattress Installation Covering the Entire Channel Perimeter

Stacked Block Gabions

Stacked block gabion revetments consist of rectangular wire baskets, which are filled with stone and stacked, in a stepped-back fashion to form the revetment surface. They are also commonly used at the toe of embankment slopes as toe walls, which help to support other upper bank revetments and prevent undermining.

5.4 Discharge Control

The river training structures are either in the flow direction or parallel to it, or they are perpendicular (or at some angle) to the flow direction.

i) Closure of secondary branches: In case of complete closure of a secondary branch, after a new equilibrium has been reached, all water and sediment will follow the remaining branch. When the final equilibrium in the new channel has been reached, a new slope, width and depth will be present.

ii) Short-cut of bends (or artificial cut-off)

When a river contains sharp bends, bend cutting may be required for several reasons.

1. to improve the situation for navigation – e.g. reduce the length
2. to improve flood discharge capacity – It may also be used to direct the river from the curved flow which may be endangering valuable land and property
3. to stop severe bank erosion
   - to straighten out the approach of the river for some structures to be constructed just below the bend

Bend cutting is executed by dredging a new channel along a much shorter but stable alignment. To get a stable channel, a gentle bend should be made.

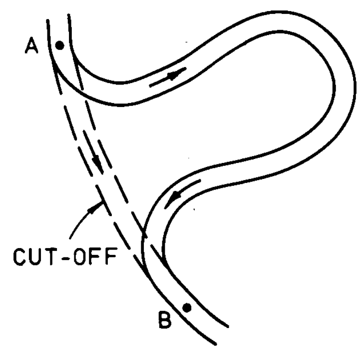


Figure 5.15 Short-cut of bends

- **Morphological consequences of short-cutting are:**
  - Energy gradient of the stream along the cut-off channel and upstream of it is increased; hence also its sediment transport capacity.

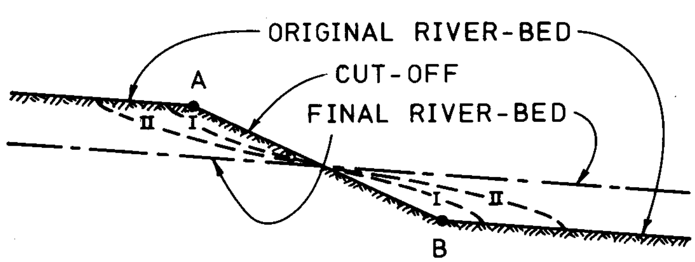


Figure 5.16 Erosion and deposition at cut-off

If the elevations at points A and B remain unaltered, the longitudinal slope between the two points along the cut-off is considerably increased. As a result, there are two transitions

i. From the mild slope upstream of A to the steeper slope of the cut-off;
ii. From the steeper slope of the cut-off to the mild slope downstream of B.

Generally, erosion starts first not far from point A and deposition will take place in the vicinity of point B. With time, the erosion moves upstream from point A (back erosion), and sedimentation advances downstream from point B. The back erosion and progressing deposition are carried on until eventually a new longitudinal slope is established, more or less similar to the original slope.

- Because of reduced stream storage, peak discharge downstream of the cut-off is likely to be higher than before. The streambed upstream of point A is lowered with time, while downstream from point B it will be raised above the original streambed before the construction of the cut-off. This reduction in channel capacity in the downstream part may cause flooding at higher discharges, since the water can no longer be contained within the stream channel. Therefore, in most cases, meander short-cutting alone is not sufficient to prevent the stream from overflowing its banks during the flood protection design discharge, hence additional means are necessary, such as channel improvement or dykes.

iii) Marginal Embankments (Dykes and Levees)

A levee or dyke is a structure mainly for flood protection by controlling the river and not by training it. The alignment should follow the normal pattern of meandering of the river. They are constructed of earth materials and may be provided at one or both sides of the river. The design of dykes or levees is just like embankment dams.

Like embankment dams they are likely to fail due to overtopping, piping, seepage, etc. They are designed to hold water up to the maximum anticipated H.F.L. without the possibility of overtopping and withstanding all external pressures. Therefore, the necessary conditions are met by providing sufficient freeboard, bed width, top width and stone protection on slopes. As the height increases it becomes necessary to provide key trenches, zoned sections, etc. to make the embankment stable. Freeboard may be between 0.3 m and 1.5 m above H.F.L.

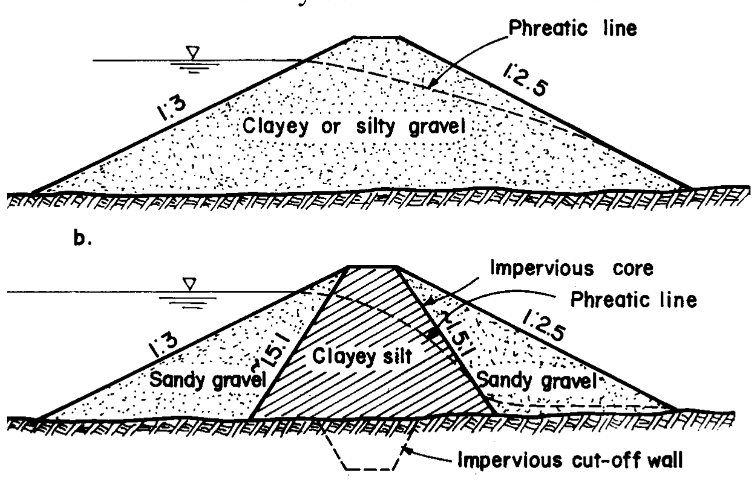


Figure 5.17 Typical dike cross-sections

It is obvious that the same discharge could be carried between higher dykes built close to the stream bank, and low dykes built away from the stream.

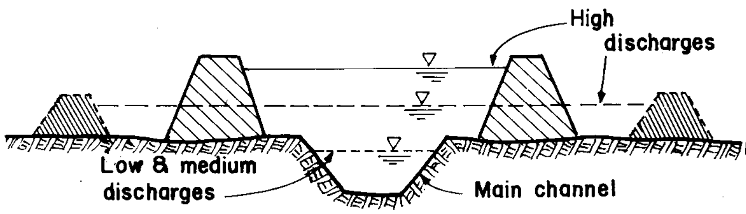


Figure 5.18 Low and high dykes

With the exception of cases in which the distance of the dykes from the stream is limited by circumstances that fall mainly in the legal province (such as property right, expropriation restrains, etc), the distance, and hence, also their height is based on consideration of:

ii. Economics: concerning cost of dykes – low dykes are cheaper to build, not only because of minor volume of earthwork but due to other construction considerations – e.g. protection against piping; clearing foundation area from vegetation, roots, boulders, or organic matter; compaction in layers; etc. Dykes are usually built along extended stretches of the stream; hence cost of dyking scheme is sensitive to additional height.

iii. Safety: Failure with low dykes will result in only minor damages, while with high dykes, both the material damage and human suffering are likely to be more severe.

iv) Guide Banks (or Guide Bunds)

Rivers in flood plains submerge very large area during flood periods. When some structure is to be constructed across such a river (e.g. bridge, weir, etc) it is very expensive to construct the work spanning the whole width of the river. Therefore, guide banks are constructed to confine the flow of the water within a reasonable waterway. They extend both upstream and downstream of the abutments of the structure. They are generally provided in pairs symmetrical in plan. Upstream curved head is provided to have a bellmouth entry and the downstream curved head to have smooth exit.

The remaining portion of the river on either side of the guide banks up to the edges or banks of the flood plain is covered by embankments known as approach banks. The guide banks guide the river flow past the bridge or any other hydraulic structure without causing damage to work and its approaches.

The main parts of a guide bank are:

1. Upstream curved head or impregnable head
2. Downstream head
3. Shank or straight portion which joins the two curved heads
4. Slope protection and launching apron

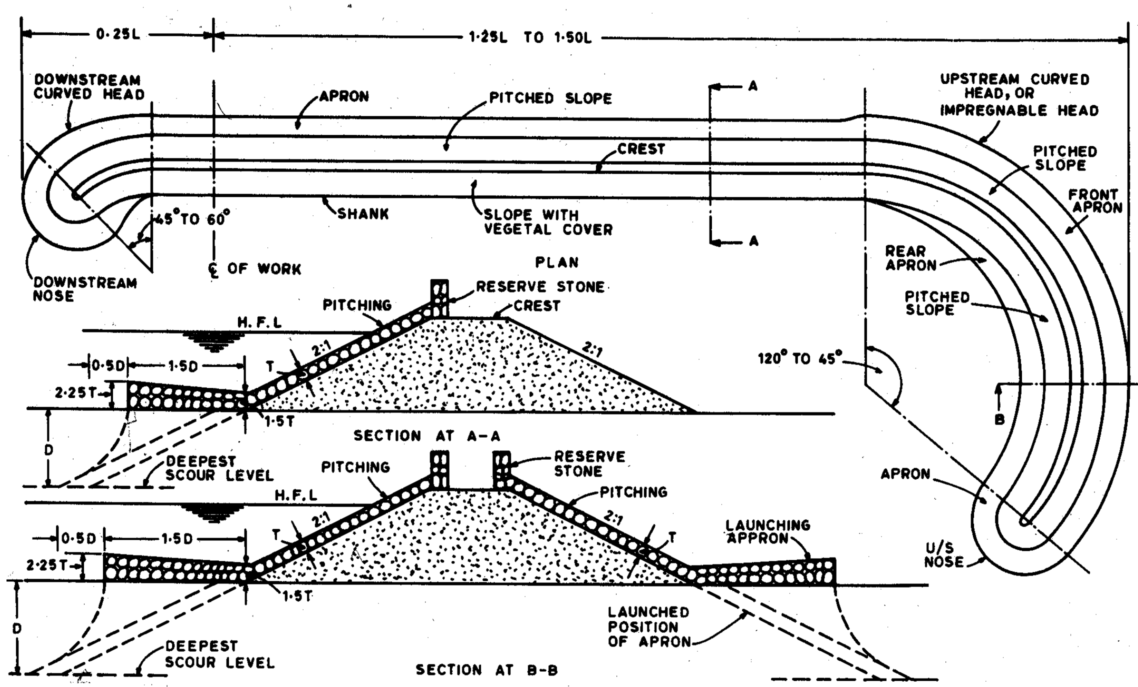
➢ **Design Criteria for Guide Banks**

i. Length of Clear Waterway: to be provided between the guide banks or the abutments of the work is given by:

$$\mathbf{P = 4.75\, Q^{1/2}}$$

where P = Lacey's Regime perimeter (m), Q = maximum discharge (m^3/s)

The length, L, of the overall waterway (or length of the work or the structure) between the guide banks or the abutments of the work is obtained by adding the thickness of the piers (in case of bridges) to P. Generally, **L≈ 1.1 to 1.25 P**.



ii) Length of Guide Bank: According to Spring, length of guide banks on the upstream side from the axis of the work should be equal to 1.1L and downstream side from the axis of the work 0.1L to 0.2L. Other formulas (e.g. Gales) depending on the discharge are also recommended:

- Upstream of axis of work: 1.25 L for Q_{max} up to 20,000 m^3/s
  1.25L – 1.5L for $21{,}000<Q_{max}<42{,}000$ m^3/s
  1.5L for $Q_{max} >42{,}000$ m^3/s

- Downstream of axis of work: 0.25L for all sizes of rivers.

iii. Radius of Curved Heads:

Upstream curved heads – sweep angle of 120° to 145°; R = 0.45L

Downstream curved heads – sweep angle of 45° to 60° and half radius of upstream curved head; $R_1 = ½ R$

(See also the sketch above)

iv. Cross section of Guide Banks:

- Top width not less than 3 m
- Constructed of locally available material, usually sand (earthen, soil)
- Side slopes not steeper than 2:1 (H:V)
- Free board of 1.25 to 1.5 m above anticipated flood level

i. Slope Protection for Guide Banks:

- Water face protected by stone pitching (each stone weighing 40 to 50 kg)
- Rear face slope provided with vegetal cover to protect it against wind & rain erosion
- At both curved ends the pitching is done on both front and rear faces
- The pitching must extend up to 1 m above the maximum flood level.

The pitching as recommended by Inglis is given by

$$\mathbf{T = 0.06Q^{1/3}}$$

where T = thickness of stone pitching (m)
Q = maximum discharge (m^3/s)

The thickness of the pitching must be 25% more at the impregnable head than the rest of the bund.

ii. Launching apron:

The slope of the guide bank may be damaged due to scour, which may occur at the toe of the bank with consequent undermining and collapse of the stone pitching. In order to protect the slope against such damage a stone cover known as launching apron is laid from the toe of the bank on the horizontal river bed, so that when scours it first undermines the apron starting at its farthest end and extending backwards towards the slope. The apron then launches to cover the face of the scour with stones forming a continuous carpet below the slope of the guide bank. In order to ensure complete protection of the whole of the scoured face, adequate quantity of stone should be provided in the launching apron.

Slope of scoured face is assumed to be 2:1 (H:V). Thus if D is the depth of scour below the bed of the river, then the length of the scour face will be (√5) D.

Depth of scour below high flood level (H.F.L.) = K R_s

Where $R_s = 0.47\left(\frac{Q}{f}\right)^{1/3}$ = *Lacey's regime scour depth (m)*

Q = maximum discharge (m^3/s)
f = silt factor

and $f = 1.75\sqrt{d}$; d = mean size of bed material (mm)

Table 5.1 Recommended scour depths

Location	Range of scour depth = KR_s	Mean scour depth to be adopted in design = KR_s
Nose of guide bank	2.00 – 2.50 R_s	2.25 R_s
Transition from nose to straight portion	1.25 – 1.75 R_s	1.5 R_s
Straight portion of guide bank	1.00 – 1.50 R_s	1.25 R_s

In launching, since the apron will not form a uniform stone carpet as by hand packing, the thickness of the apron in launched position is assumed to be 1.25T. Thus, the quantity of stone required per meter length of the launching apron will be

$$(\sqrt{5})\, D \times 1.25T = 2.80\, TD$$

Width of launching apron = 1.5D (usually).

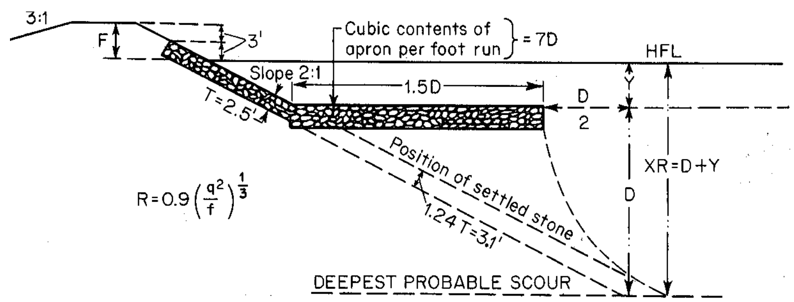


Figure 10.19 Launching apron

Example: The following hydraulic data pertains to a bridge site of a river:

$Q_{max} = 6000\ m^3/s$

H.F.L. = 104 m

River bed level = 100 m

Average diameter of bed material = 0.10 m

Design and sketch a guide bund including the launching apron to train the river.