RIVER TRAINING and BANK PROTECTION

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The various studies carried out by the Bureau of Flood Control and Water Resources Development of the Economic Commission for Asia and the Far East are being published as *Flood Control Series.* The first number of this series entitled "Flood Damage and Flood Control Activities in Asia and the Far East" was published in October 1950. The second volume of the series entitled "Methods and Problems of Flood Control in Asia and the Far East" was published in 1951. The third number, "Proceedings of the Regional Technical Conference on Flood Control in Asia and the Far East", was published in 1952. The present publication "River Training and Bank Protection" is the fourth one of this series.

This study deals with the general theory and principles of river training and river bank protection. A description is given of the various methods of river training and bank protection that are adopted in the different countries of Asia and the Far East as well as in Australia, Europe, New Zealand, and the United States of America. Recommendations have been made on the basis of the comparison of different methods in use. A questionnaire on river training and bank protection has been appended with a view to collecting further information on the subject.

The fifth number of this series entitled "The Sediment Problem" will be published shortly.

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SYMBOLS

The following symbols have been used in this publication to represent the quantities mentioned against each, unless otherwise stated inside the text of the publication.

A	Area
B	Surface width of a channel
b	Sub-script for bed
C, C_1, C_2	Coefficients
c	Concentration of sediment by wt. or vol. per unit wt. or vol. of fluid
D	Depth
D_m	Mean depth
D_s	Depth of scour
d	Diameter of sediment particles
F	Force
f	Silt factor
g	Gravitational constant
H	Head of water
K	A constant, a ratio
l	Length
m	An exponent
N	Manning's roughness coefficient
P	Wetted perimeter
Q	Total discharge of water
Q_s	Rate of sediment transport
q	Discharge of water per unit width of channel
q_s	Rate of transport of sediment per unit width
R	Hydraulic mean radius
R_e	Reynolds' Number
S	Slope
s_s	Specific gravity of sediment
s	Distance travelled by sediment particles
T	Thickness
t	Time interval
V	Velocity of flow
V_0	Critical (non-scouring and non-silting) velocity
W	Bottom width of a channel
W_d	Sediment run-off
w	Terminal fall velocity
x, y, z	The co-ordinate axis
α_1, α_2, α_3,	Coefficients
β	Constant
ϕ	Angle of side slope
θ	Angle of repose
ϕ_1, ϕ_2, ϕ_3	Coefficients
μ	Absolute viscosity
ν	Kinematic viscosity
τ	Tractive force
τ_c	Critical tractive force
γ	Specific weight

Chapter I

INTRODUCTION

ORIGIN OF THE STUDY

Since time immemorial embankments have been, and still remain, the most important measure of flood control in Asia and the Far East. Over 20,000 km. of embankments have come into existence, protecting a population of 250 million, or approximately 22 per cent of the total population of Asia and the Far East region.

Failure of embankments may lead to catastrophic damage to land and people and is quite a common occurrence on many rivers in the region. One of the main reasons of their failure is bank erosion — the washing away of an embankment by flood water or the washing away of river banks followed by a failure of embankment. Successful protection of river banks and embankments against erosion either by proper protection or by river training, is, therefore, considered essential to the well-being of the people of the region.

On the proposal of the Bureau of Flood Control and Water Resources Development, the Economic Commission for Asia and the Far East, at its seventh session held at Lahore, Pakistan in March 1951, approved the programme of work of the Bureau to include a joint study on river training and river bank protection with teachnical organizations of the region.

A preliminary draft was prepared by the Bureau after a careful review of the technical information made available by the various technical organizations and the field investigations conducted by experts of the Bureau. This draft was submitted to the various technical organizations for their comments and suggestions[1] before it took its final form as printed in this book.

SCOPE OF THE STUDY

The expression "river training," in its broader sense, implies any construction provided on a river to direct and guide the river flow including floods, to train and regulate the river bed or to increase the low water depth. In this study, emphasis is given, however, to the commonly accepted concept of river training in a restricted sense—that is, the training of rivers attempted mainly to change the configuration of their beds, though the guiding of flood flow by means of embankments and the training of low-water channels to increase the depth for navigation purposes will be briefly discussed. Dams or sluices constructed across a river, having an appreciable effect on the river bed but not designed for the improvement of the latter, are not included in this study; whereas cut-offs, being a measure directly connected with river bed, will be included under river training.

When one considers that bank revetment is an indispensable part of river training and that some transverse type of river training works may be used invariably as bank protection works, there can be no marked line of demarcation between the two. However, as the aim of bank protection is generally very limited, there is no harm in treating it as a separate subject, taking care not to duplicate its discussion elsewhere. Thus bank revetment for the purpose of protection and indirectly deflecting the flow will be discussed under bank protection and transverse structures, provided with the object of deflecting the flow away from the banks and indirectly performing the task of bank protection, will be included in the subject of river training.

The sediment problem is a subject closely related to river training and bank protection, but in itself presents a vast problem needing

1. The Bureau gratefully acknowledges the valuable comments and suggestions received from the following organizations and individuals: The US Waterways Experiment Station, Vicksburg, Mississippi, USA; The US Bureau of Reclamation; Rigkswaterstaat Directie van den Waterstaat, Netherlands: The Central Water and Power Commission, India; The Ministry of Construction, Japan; and also from Mesrs. L. N. McClellan, Thomas Maddock Jr., L. Venkatakrishna Ayyar, N. K. Bose, A. Normandin, A. G. Maris and E. W. Lane.

comprehensive treatment in a separate report.[1] Only that part of the sediment problem which has a direct bearing on river training and bank protection, such as the transportation of sediment in rivers, the scouring of river beds etc. will be briefly dealt with in this study.

DEFINITION OF TERMS

As many engineering terms derive their names from local use and may differ widely from one region to another, the writing of engineering reports sometimes becomes confusing and even misleading; the terms are not clearly defined and their use is not consistent. As mentioned before, an engineering term like "river training" used loosely may include a whole treatise on hydraulic structures. Definitions to limit the scope of each term, stating the difference from ordinary synonyms are, therefore, highly desirable. The following terms will be used throughout the text unless direct quotations are made. Whenever special names for certain types of construction derived from local use are followed, a proper description of these will be given.

(1) *Bank protection.* Bank protection is the kind of engineering work which aims at the protection of banks of a river, or slopes of embankments along it, from erosion by the current of flow.

(2) *Bank revetment.* Bank revetment is a type of bank protection which covers continuously the entire slope of a bank or an embankment, including the portions extending far into the river bed, to keep the bank from receding landward owing to erosion. Bank revetment somewhat resembles canal lining in appearance; the difference between them is that the latter aims at stopping or lessening the seepage of water from a canal into the ground, while the former is not much concerned with the seepage flow, except in regard to the drainage of underground water from the land to the river, which may be harmful to the stability of the banks.

1. Bureau of Flood Control, Economic Commission for Asia and the Far East, *The Sediment Problem*, Flood Control Series No. 5.

(3) *Bunding.* Bunding is a kind of bank protection where, owing to space or usage, or for strength, a continuous vertical or nearly vertical wall or other structure is built in front of the margin.[2]

(4) *Embankment.* An embankment is an artificial bank built along a river for the purpose of protecting adjacent land from inundation by flood waters. It is called "levee" or dike" in different countries and called "bund" in India.

(5) *River training.* River training is a group engineering work including artificial plantations, with or without the construction of embankments, built along a river or a section thereof in order to direct or to lead the flow into a prescribed channel.

(6) *Groyne or bankhead.* A groyne or bankhead is a structure built from the bank of a river in a direction transverse to the current.

(7) *Training wall.* A training wall is a structure built along or connected to the bank of a river parallel to the direction of flow.

(8) *Sill.* A sill is a structure built under water, across the deep pools of a river, with the aim of correcting the depth of a river.

(9) *Closing dike.* A closing dike is a structure built across the branch channel of a river in order to stop or reduce the flow entering that channel.

2. "Bunding" is a word derived from "bund" which is extensively used in India as a synonym for the word "embankment", but in the Far East its meaning is limited to a quay or a riverside drive with practically vertical structures to retain the earth.

Chapter II

RIVER TRAINING

CLASSIFICATION OF RIVERS

Rivers can usually be divided, according to the topography of the river basin, into two parts: the upper reaches in the hilly region and the lower reaches on the alluvial plain. Rivers in the hilly region are characterized by the steepness of their slope, the swiftness of the flow, the occurrence of land slides and the formation of rapids along their courses. The control of rivers in the head reaches is known as "torrent control" and the methods adopted are distinctly different from those applied on alluvial rivers known as river training.

Alluvial rivers are characterized by the fact that the alluvia, on which the rivers flow, are built up by the rivers themselves. Depending upon the material carried by the rivers, the alluvia may be composed of fine silt over large areas such as the alluvial plains of the Yellow River, Yangtze, Red River, Mekong, Chao Phya, Irrawaddy, Ganga and the Indus; or of shingles and gravels such as the small strips of alluvia of rivers near the coast in islands of Japan and Taiwan.

Rivers on alluvial plains may be broadly classified into three types: (*a*) the meandering type, (*b*) the aggrading type, and (*c*) the degrading type. A river on an alluvial plain is seldom of a single type, but all three types may be found on the same river from its uppermost point on the alluvial plain to its mouth. The classification may solely depend upon the amount and the size of sediment entering the river and its carrying capacity for the sediment load. As the sediment load and discharge vary with respect to time, any particular section of a river may be aggrading, degrading, or meandering at different times.

Generally speaking, where a river has enough capacity to carry the incoming sediment downstream without forming large deposits, the whole or a part of it would be of the meandering type. On the other hand, owing to excessive sediment entering a river with sudden diminution of slope on the plain, or to some obstruction like a barrage or a dam across it, which raises the level of water and flattens the slope, or owing to the extension of the delta at the river mouth, or to sudden inrushing of sediment from a tributary, the river will build up its bed to a certain slope and is of an aggrading type. Instead of meandering, a river of aggrading type, or a section of it, has usually a straight and wide reach with shoals in the middle and divided flow, which may even result in braided flow and an intricately woven system of channels. The third or degrading type of river, or rather section of a river, is found above a cut-off or below a dam or a barrage, and results either from the sudden lowering of water surface due to the cut-off which increases its slope of flow, or from the sudden diminution of its sediment load caused by the damming up of the water above.[1] For the same reason, one can expect a meandering type of river to develop temporarily into a degrading type where river training or bank protection is carried out to such an extent that there is no further appreciable erosion of the banks and consequently a reduction of sediment load takes place.[2]

1. From a theoretical point of view in regard to the dynamics of rivers, it may be said that rivers have different ways of adjusting their alignment, cross-section and slope according to the particular combination of discharge and sediment load prevailing. Aggrading, degrading and meandering are the various ways of adjustment.

2. From a comprehensive series of experiments conducted in the US Waterways Experiment Station at Vicksburg, Friedkin found that a river tends to be shallow and braided if the bank material is highly erodible, and tends to be deep and uniform if the bank material is highly resistant (see Friedkin, J. F., *A Laboratory Study of the Meandering of Alluvial Rivers*, Vicksburg, May 1954. This problem can perhaps be further analysed in the light of the boundary shear and velocity distribution in a cross-section of the river and the resistance of material of the of the river bank and bed. The boundary shear distribution of a trapezoidal channel has been worked out by Olsen and Florey (see US Bureau of Reclamation, *Structural Laboratory Report No. SP-34*, August 1952). According to Olsen Florey, the maximum boundary shear occurring at the side of a trapezoidal channel, having a side slope of 1½ to 1 and a bottom width eight times the depth, is 0.767 of the maximum shear at the bottom of the channel. This means that the acting force of running water on the side of a trapezoidal channel is definitely smaller than that acting on the bottom of the channel. If the resistance of river bank material and that of river bed material composed of clayey material with strong cohesion are the same, the river will be subject to downward scouring as the acting force is larger at the bottom than on the side. If the river bed and bank are composed of non-cohesive sandy material, the resistance of cohesionless sandy material of the bank is smaller than that of the bed, as the former, being on an inclined plane, is also subject to sliding. In a trapezoidal channel with side slopes mentioned above, if the ratio of resistance of the material on the bank to that on the bed is smaller than 0.767, lateral erosion will take place and the river will tend to become braided and shallow. The problem is further complicated by the transportation of of that sediment which usually moves along the bed and "consumes" part of the erosive energy of running water which acts on the bed. Readers are referred to Flood Control Series No. 5 *The Sediment Problem* published by the Bureau of Flood Control, in which some numerical examples of boundary shear distribution and erosion have been given.

We may say that among the three types of rivers mentioned above, the meandering type is the full and final stage of river development, while the other two are interim types which might be maintained as long as the same causes and effects remain unchanged. It will also be seen that the final or meandering stage of river development is by no means permanent, as any change in the balance, caused by natural or artificial forces, would affect its regime. But there is a definite regime for each river to follow, with its distance and elevation from the source to the sea, the magnitude and variation of its discharge and sediment load and the composition of the soil composing its bed and banks. Although these elements of river flow are in no sense permanent, and some vary considerably from year to year, yet there is a limit which controls the general dimensions of a river. The training of a river is meant to confine it within this limit for efficient flow and therefore the type to be followed for river training should be the fully developed final stage of the river, i.e. the meandering type.

PURPOSE OF RIVER TRAINING

River training in its modern sense started in Europe in the last century, for the improvement of navigable rivers, with the object of maintaining a better and deeper channel for navigation which had been expanding along with industrial development at that time; however, training of a river by embankments along its banks, in order to confine the flow in a single deep channel to carry away the sediment burden to the sea, had been practised on the Yellow River in China as early as the end of the 16th century.

Later on, the construction of rail and road bridges and irrigation headworks on Indian rivers, which are usually shallow and wide, necessitated the narrowing and training of certain sections of the rivers to direct the flow through bridge openings or over weirs or to regulators.

In some projects of flood control, for example, in the Miami Conservancy District of Ohio, USA, certain sections of the rivers had to be trained or improved for increasing the channel capacity in discharging floods. Cut-offs were carried out on the Lower Mississippi with effective shortening of the river course, thus reducing both the flood heights above the cut-off and the flood period on the entire river.

H. Engels and O. Franzius in Europe, and the Waterways Experiment Station at Vicksburg, Miss., USA,[1] demonstrated by model experiments that the stabilization of a river channel would result in lowering of its bed. The new scheme of the Mississippi, which comprised revetting of 44½ per cent of its bank and the stabilization schemes of the Yungting and the Yellow rivers in China, are interesting examples.

Briefly, the purpose of river training is to stabilize the channel along a certain alignment with certain cross-section for one or more of the following objects:

(1) Safe and expeditious passage of flood flow
(2) Efficient transportation of suspended and bed load
(3) Stable river course with minimum bank erosion
(4) Sufficient depth and good course for navigation
(5) Direction of flow through a certain defined stretch of the river.

THEORY OF RIVER FLOW AND RIVER TRAINING

Classification of river training

In accordance with the object to be achieved, river training works can be classified into (*a*) high water training, (*b*) low water training and (*c*) mean water training.

(*a*) *High water training*

High water training aims at the provision of sufficient and efficient cross-sectional area for the expeditious passage of the maximum flood. It concerns essentially the alignment and height of embankments for a given flood discharge. In general no attempt is made to change the condition of the river bed although cut-off and dredging are carried out in some sections. High water training can be called *Training for discharge.*

(*b*) *Low water training*

Low water training aims at the provision of sufficient depth in the channel during low water period for navigation. This is usually achieved by contraction of the width of the channel. Low water training can be called *Training for depth.*

1. National Economic Council, China, *Model Tests on the Regulation of the Yellow River* by H. Engels (in German), Nanking, 1936 and various publications of the US Waterway Experiment Station, Vicksburg.

(*c*) *Mean water training*

Of the various kinds of training, mean water training is the most important. Any attempt to change the configuration of the river bed, in alignment or in cross-section, should be designed in accordance with that stage of flow when the total work done with respect to bed building in a year or over a period of years is the maximum. It will be shown later (see page 16-17) that maximum bed building stage is somewhere in the neighbourhood of mean water.[1] Mean water training therefore forms the basis on which both the high and low water training are to be planned. As mean water training aims at the correction of the configuration of the river bed and the efficient transport of sediment load to keep the channel in good shape, it can be called *Training for sediment*.

Meandering of rivers

As pointed out before, among the three types of alluvial rivers, viz. the aggrading, the degrading and the meandering type, the last type indicates the fully developed final stage of an alluvial river, which is generally adopted in the training of rivers. The phenomenon of river meandering was observed and described, a long time ago, by many river engineers in Europe and America, as well as by irrigation engineers in India; but the most elaborate contribution on this subject must be attributed to Fargue,[2] a French engineer who worked on the river Garonne, from which he derived the well-known Fargue's laws for river training. Fargue's contribution has been very significant and is a guide for river engineers even up to the present day. Whether the cause of meandering is as simple as concluded by J. F. Friedkin,[3] from his experiments at Vicksburg, Miss., or is due to excessive charge of bed-load in movement and the development of flow attracted towards one bank as described and discussed by Inglis and Joglekar[4] in their monumental work, it is a fact, never disputed amongst engineers, that the meandering of a river, once established, has a certain pattern and dimensions, varying acording to the discharge, slope and composition of materials of the bed and the banks of the river. This fact has been observed both on natural rivers and on models, although no quantitative relations have yet been arrived at.

The conclusions reached by various experimenters on the meandering of rivers are summarized below:

(*a*) A river meandering through an alluvial plain must form a series of consecutive curves of reversed order connected with short straight stretches called crossings, with the maximum depth at or near the apex of the bend and gradually diminishing to the shallowest section at the crossing. A meander, when fully developed, with a homogeneous valley material and a constant discharge, shall have a definite pattern of curvature, length, width and depth of channel which is reproducible in models. The work of Inglis and Joglekar indicates that the width of the meander belt (the transverse distance between the apex point of one curve and the apex point of the reverse curve) and the length of meander (the air distance along the river measured from the tangent point of one curve to the tangent point of the curve of same order), as well as the width of the river, all vary roughly as the square root of the discharge.

(*b*) The pattern of meander changes whenever there is a change in discharge, slope or bank and bed forming material. Generally speaking, the size of bends, their length and width, as well as the degree of sinuosity (the ratio between the talweg length and the air line distance), all increase with increase in discharge or slope. Increase of sediment charge or increased rate of bank erosion increases the slope as well as the width of the river, while the depth of the river is reduced.

(*c*) Every phase of meandering thus depends upon the combined influence of three closely related factors: (*i*) the discharge and hydraulic properties of the channel; (*ii*) the sediment charge of the stream, and (*iii*) the relative erodibility of the bed and banks. Their inter-relationship is very complex because no single variable can be considered fully independent of the others.

The tractive force theory[5]

It has been pointed out before that mean water training aims at the correction of the configuration of the river bed into certain alignment and cross-section in accordance with the pattern of a well developed meandering river. As all alluvial rivers carry sediments in them, both in suspension and as bed load, it is highly essential that the channel should be designed in accordance, not only with the flow of water, but also with the flow of sediment. The law governing the design of channels for discharge

1. Strictly speaking, the bed building stage varies with the form of the stage frequency distribution curve. In some torrential rivers where the flow increases suddenly from low water to floods and back to low water without having a pronounced period of mean water, the bed building stage is somewhat in the neighbourhood of high water. This is the case of the Yellow river in China, rivers in Japan and some Himalayan rivers in India.

2. Fargue, M. *La forme du lit des rivières à fond mobile*, 1908.

3. Friedkin, J.F., *A Laboratory Study of the Meandering of Alluvial Rivers*. US Waterways Experiment Station, Vicksburg, Miss., 1 May, 1945.

4. Inglis, C. C. and Joglekar, D. V. *The Behaviour and Control of Rivers and Canals* (Government of India, Central Water and Power Research Station, Poona, 1944), p. 143.

5. See also Bureau of Flood Control, ECAFE, Flood Control Series No. 5, *The Sediment Problem*, Bangkok, 1953.

alone is quite well established but the law governing the design of channels for sediment is still at the stage of "cut and try."

Sediment load in rivers is usually divided into bed load and suspended load, with the former denoting particles sliding, rolling and hopping on or near the bed and the latter denoting those suspended in water. Strictly speaking suspended load movement is but an advanced stage of bed load movement and no sharp demarcation can be made between the two, particularly in the region near the river bed, where the sediment particles are in a stage of "saltation". However, it is for the convenience of discussison that a distinction is usually made and the portion of sediment hopping or saltating near the bed, which might be considered as the lower part of suspended load in a verticle, is generally taken as bed load.

As to which one of the two loads, whether the suspended or the bed load, is more important to the formation of a river bed this is still an open question. Results of experiments on suspended sediment indicate that if sediment particles are kept truly in suspension, the hydraulic properties of suspended flow are very much the same as those of clear water flow.[1] If a stream could discharge the whole of its suspended fine sediment into the sea, there would be no reason to believe that suspended load would affect the configuration of river bed if there is neither deposition nor erosion. The heavy silt flow of the Yellow River is a typical case of suspended sediment flow in which deposition takes place as a result of the fact that the river is not capable of carrying all of its suspended load into the sea. Generally speaking, it is the bed load including that part of suspended load which is very near to the river bed and is subject to deposition and pick-up that decides the configuration of the river bed.

In this and the following sections, an attempt is made to describe two theories used as basis for design, i.e. the tractive force theory and the regime theory. Both these theories are developed for straight channels of uniform depth only. The quantitative effect of winding of river course on sediment flow has not yet been established, but the qualitative effect will be discussed briefly.

(*a*) *Critical tractive force*

For uniform flow, the mean force of running water exerted on the river bed in the direction of flow is equal to

$$\tau = \gamma RS \qquad (1)$$

where τ is the mean tractive force per unit area of the river bed, γ the unit weight of water, R the hydraulic radius, and S the slope of river. River bed material offers resistance to the tractive force exerted by the running water. When the tractive force is equal to, or greater than the resistance of the bed material, bed material will be set in motion. The force corresponding to this critical stage is called the "critical tractive force" τ_c. For non-cohesive material, Shields[2] found by systematic experiments the relation of critical tractive force to river bed material as follows:

$$\tau_c = \phi \left(\frac{d\sqrt{gRS}}{\nu} \right) \gamma (s_s - 1)\, d \qquad (2)$$

where d is the mean diameter and s_s the specific gravity of the river bed material, g the gravitational constant, and ν the kinematic viscosity of water. The functional relation of the variables, based on experiments of uniform size bed material as carried out by Shields, is shown in figure 1. For rather high values of $\frac{d\sqrt{gRS}}{\nu}$, which is nearer to river flow, the function ϕ approaches a constant of 0.06 and its lower limit is around 0.04. More recent experiments by Meyer-Peter and Müller[3] covering particle sizes from 0.4 to 30 mm and specific gravities from 1.25 to 4.2 gives the value of ϕ for beginning of bed load movement as 0.047. Taking this value and the average specific gravity of river bed material as 2.65 and expressing the critical tractive force in kg/m^2 and the mean diameter of particles in mm, the equation takes the form

$$\tau_c = 0.078d \qquad (3)$$

From a study of the experimental results of various authors and studies made on the San Luis Valley, USA, Lane[4] found the constant in the above formula to be 0.071, corresponding to $\phi = 0.043$, which is very close to the figure given by Meyer-Peter and Müller. Table 1 is re-arranged from the tentative values of critical tractive force and velocity for different kinds of bed material as suggested by Lane (see also figure 2). This formula is very useful, for example, in the investigation of artificial cut-offs to determine whether the tractive force of the flow in the pilot channel would be sufficient to scour the bed, the composition of bed material being known.

1. Bureau of Flood Control, ECAFE, Flood Control Series No. 5, *The Sediment Problem*, Bangkok, 1953.

2. Shields, A., *Anwendung der Aehnlichkeitsmechanik und der Turbulenzforschung auf die Geschiebebewegung*, Mitt. Preuss. Versuchsanstalt f. Wasserbau und Schiffbau, Berlin, Heft 26, 1936.

3. Meyer-Peter, E., and müller R., "Formulas for Bed-load Transport", in *Proceedings of the Second Meeting of the International Association of Hydraulic Structures Research*, p. 57.

4. Lane, E. W., *The Prevention of Scour in Earth Canals*, May 1951 (Mimeographed).

(*b*) *Effect of meandering of channel on tractive force*

For flow over bends, both the shifting of maximum velocity towards the concave bank and the development of secondary current tend to increase the tractive force of the flow as given by the expression γRS. For a given bed material in a winding river course, less tractive force is required to set it in motion as compared with a straight channel. Lane therefore suggested a reduction of critical tractive force for different degrees of "sinuosity" of channels and his proposed values are given in table 2. This table is intended to be used for the design of irrigation canals only, but it might be considered as a good reference for meandering rivers.

Table 1

Critical tractive force and velocity for different bed materials (after E. W. Lane)[1]

Material	Approximate size interval[2] mm.	Approximate size interval[2] in.	Critical tractive forces (kg/m^2)	Approximate velocity m/sec	Manning's coefficient of roughness
Sandy loam (non-colloidal)	Smaller than 0.062		0.20	0.50	0.020
Silt loam (non-colloidal)	Smaller than 0.062		0.25	0.60	0.020
Alluvial silt (non-colloidal)	Smaller than 0.062		0.25	0.60	0.020
Ordinary firm loam	Smaller than 0.062		0.37	0.75	0.020
Volcanic ash	Smaller than 0.062		0.37	0.75	0.020
Stiff clay (very colloidal)	Smaller than 0.062		1.22	1.15	0.025
Alluvial silts (colloidal)	Smaller than 0.062		1.22	1.15	0.025
Shales and hard-pans	Smaller than 0.062		3.18	1.85	0.025
Fine sand (non-colloidal)	0.602—0.25		0.12	0.45	0.020
Medium sand (non-colloidal)	0.25 —0.5		0.17	0.50	0.020
Coarse sand (non-colloidal)	0.5 —2.0		0.25	0.60	0.020
Fine gravel	4—8	0.08—0.3	0.37	0.75	0.020
Coarse gravel	8—64	0.3 —2.5	1.47	1.25	0.025
Cobbles and shingles	64—256	2.5—10	4.40	1.55	0.035
Graded loam & cobbles (non-colloidal)	0.004—64		1.96	1.15	0.30
Granded silts to cobbles (colloidal)	0—64		2.20	1.25	0.30

Table 2

Critical tractive force in sinuous canals (after E. W. Lane)

Degree of sinuosity	Percentage of critical tractive force required to initiate bed load movement as compared with straight canals	Corresponding percentage of mean velocity
Straight canals	100	100
Slightly sinuous canals	90	95
Moderates inuous canals	75	81
Very sinuous canals	60	78

1. Computed from "The Permissible Canal Velocities" by Fortier & Scobey, in *Transactions of the American Society of Civil Engineers*, vol. 89, 1926, p. 955, using the roughness coefficient shown.

2. *American Geophysical Union, Transaction*, vol. 28, Number 6, December 1947, p. 937.

(c) *Rate of sediment transportation*

When the actual tractive force of river flow exceeds the critical value, river bed material will be set in motion. The rate of bed load transportation depends upon the magnitude of the prevailing tractive force (in excess of the critical value) of the flow. Naturally, the rate of transportation increases with the increase of stage or rather the discharge of the river. For the same stage of water level, the rate of sediment transportation as well as the discharge are different for rising or falling stage of flow.

It is important to find out the rate of transportation of bed load for different stages of flow. Actual measurement, taken at the river bed with some appropriate bed load samplers during different stages of flow, is desirable although difficult. No apparatus has yet been developed that can truly and conveniently measure the rate of bed load transportation. Samplers used at present usually obstruct the natural flow and tend to set up turbulent eddies in their immediate vicinity changing the rate of transport near the bed, so that the resulting sample becomes non-representative. The results of measurements thus obtained must be carefully examined and plotted in a rating curve of bed load transportation, showing the rate of transportation, in kg/sec or m³/sec over the entire cross-section in relation to river stage. This is usually possible for smaller streams and extremely difficult, if not impossible, for large rivers. A typical example of sediment rating curve of river Weichsel at Thorn is given in figure 3. As natural rivers have always winding courses, only actual measurement of bed load transportation will include the effect of winding of the river course on bed load transportation.

If actual measurement is not available, the rate of transportation may be calculated by using the formulae obtained from experiments conducted in artificial flumes, but verified by measurements carried out in natural streams as most of the formulae are based on flume tests and their extrapolation to natural rivers may introduce large errors.

Mention may be made of the bed load function developed by H. Einstein[1] which is based on experiments conducted in flumes as well as measurements taken in two small streams. His bed load function, re-arranged by H. Rouse,[2] takes the following form:

$$\frac{q_s\, s_s}{\sqrt{g\,(s_s-1)}\; F d^{3/2}} = 40 \left(\frac{\tau}{\gamma\,(s_s-1)\, d} \right)^3 \quad (4)$$

q_s = rate of bed load transportation in volume per unit width of channel

s_s = specific weight of sediment

d = mean diameter of sediment

g = gravitational constant

τ = mean tractive force of running water

γ = unit weight of water

$$F = \sqrt{\frac{2}{3} + \frac{36\nu^2}{gd^3\,(s_s-1)}} - \sqrt{\frac{36\nu^2}{gd^3\,(s_s-1)}}$$

ν = kinematic viscosity of water

The above equation is dimensionally homogeneous. For sediments with mean diameter larger than 0.2 mm, the value $\frac{36\nu^2}{gd^3\,(s_s-1)}$ is small and can be neglected as compared with $\frac{2}{3}$ in the expression *F*. Taking the specific gravity of sediment s_s as 2.65 and kinematic viscosity of water $\nu = \frac{1}{10^6}$ m²/sec, the equation becomes

$$q_s = \frac{1}{3{,}010} \; \frac{\tau^3}{d^{1.5}} \quad (5)$$

if q_s is expressed in m³/sec/m, τ in kg/m², and d in mm; and

$$q_s = \frac{1}{32{,}500} \cdot \frac{\tau^3}{d^{1.5}} \quad (6)$$

if q_s is expressed in cu ft/sec/ft, τ in lb/ft² and d in mm.

Application of the tractive force theory to river training

If the rate of bed load transportation for different stages of flow is known, the theory of tractive force could be applied, in some cases, to river training. Some graphical methods as suggested by Schaffernak[3] are briefly given below.

(a) *Determination of the "bed building" stage of rivers*

On the left hand side of figure 4, the rate of bed load transportation, in kg or m³ per second or per day is plotted against the river stage or water level with the latter as the ordinate. Using the same ordinate, the frequency of occurrence of different stages within a year or a normal year,

1. Einstein, H. "Formulas for the transportation of bed load", in *Transactions of the American Society of Civil Engineers*, vol. 107, 1942, pp. 561-577.

2. Rouse, H., *Engineering Hydraulics* (John Wiley & Son, N. Y. 1950), p. 103.

3. Shaffernak, F., *Flussmorphologie und Flussbau*, Vienna, Springer Verlag, 1950.

expressed in number of days or any other unit, is plotted on the right hand side of the figure. For a certain stage D_1, the rate of transportation is Q_{s1} and the time interval corresponding to this stage of flow is Δt_1. The product of Q_{s1} Δt_1 gives the sediment "run-off", in kg or m^3, at stage D_1. Similarly the sediment run-off at other stages is also computed and a sediment run-off curve is plotted on the left hand side of the figure using some convenient scale of Q_s Δt as the abscissa. The river stage corresponding to the maximum of the sediment run-off curve is the bed building stage of a river. Usually the bed building stage is somewhat in the neighbourhood of mean water. In order to achieve the result of river training as quickly as possible, the height of training works is usually designed in accordance with a level corresponding to the bed building or mean water stage.

(*b*) *Determination of the annual sediment run-off*

The sediment run-off can be obtained from the bed load transportation curve and the water level duration curve, as shown in figure 5. The flow duration curve is plotted on the upper right quadrant of the figure expressing, usually in days, the total time within a year in which the water level is equalled or is greater than a certain value. Starting from a certain point *a* in the duration curve, the corresponding point *c* of the sediment duration curve can be obtained by routing the guide lines through *b* of the transportation curve as shown in figure 4. The shaded area enclosed by the sediment duration curve and the co-ordinates represents the total sediment run-off within the year.

(*c*) *Effect of channel contraction on sediment run-off*

The sediment transportation curve and the sediment duration curve of the original river channel are shown in solid lines in figure 6. On the lower left quadrant of the figure, the discharge rating curve of the original channel is also shown. Experience seems to indicate that after the channel is contracted, the discharge rating curve takes a different form, having higher stages for relatively higher discharges and lower stages for the lower discharges. It is evident, therefore, that the old and new rating curves must cross at a common point on a common plotting (see figure 6). Very often the point of intersection of these two curves cannot be determined precisely but only within rather wide limits, owing to the uncertainty of the assumption for hydraulic computations and to the scatter of plotted river measurements. For a certain discharge Q, the new channel will have a higher rate of sediment transportation corresponding to the new rating curve. A new sediment duration curve can be obtained by seeing that the rate of transportation Q_{s_2} corresponds to the D_2 with the duration corresponding to D_1, as the discharge is the same. This new sediment duration curve is shown in the figure marked with (2). The shaded area lying between the two sediment duration curves gives the increased sediment run-off within a year as affected by channel contraction.

(*d*) *Effect of withdrawal of discharge from river*

In many irrigation and water power projects, a part of the clear water discharge is drawn from the river without the construction of weirs or barrages across the river. The river is then required to carry the same amount of bed load with a reduced flow of water. To investigate the possible aggradation of river bed below the intake, a method similar to above can be applied. The rating curve, sediment transportation curve and the sediment run-off curve of the original channel are presented in figure 7. The quantity of discharge withdrawn is Q_e, and a new Q–Q_e curve is drawn in the figure by laying off, at every stage D, the amount of Q_e from the original discharge Q_1. In the original condition, the discharge is Q_1, the rate of transportation Q_{s1} and the duration $\Sigma\, \Delta t_1$, obtained by running lines as shown by a-c-d-e. In the new condition, the discharges is $Q_1 - Q_e$ and the corresponding stage is D_2 with a rate of transportation Q_{s_2}. The durations for both cases are, however, the same. A new sediment run-off curve is obtained by running lines as shown by a-b-f-g-h. The shaded area, bounded by the two run-off curves, gives the total amount of sediment expected to be accumulated on the river bed within the period under consideration.

If it is intended to apply the method of channel contraction to remedy the possible aggradation due to the withdrawal of discharge, the method described in (*c*) could be applied in which a contracted cross-section is to be chosen, by cut and try, so that the increase of sediment run-off will be equal to the aggradation caused by the withdrawal of discharge.

The regime theory

In the design of irrigation canals in India and Pakistan, where the flow in the canals contains a substantial amount of sediment derived from the river flow, great care has been given to the stability of canals so that neither silting nor scouring should take place. Extensive research has been carried out on this subject as a result of which the regime theory has been developed.

Among many workers who contributed to the development of this theory, most important is perhaps Lacey[1] who, having defined the relation between many hydraulic variables, introduced a silt factor f to define the sediment. The set of Lacey's formulae is given below:

$$P = 2.668\, Q^{\frac{1}{2}} \qquad (7)$$

$$R = 0.4725 \frac{Q^{1/3}}{f^{1/3}} \qquad (8)$$

$$A = 1.26 \frac{Q^{5/6}}{f^{1/3}} \qquad (9)$$

$$V = 0.794 \frac{Q^{1/6}}{f^{1/3}} \qquad (10)$$

$$S = 0.000547 \frac{f^{5/3}}{Q^{1/6}} \qquad (11)$$

$$f = 8\sqrt{d} \qquad (12)$$

where P is the wetted perimeter in ft, R the hydraulic radius in ft, A the cross-sectional area in ft^2, V the velocity in ft/sec, Q the discharge in cu ft/sec, S the slope, f the silt factor and d the mean diameter of sediment in mm.

Bose and Malhotra, after several years of painstaking collection and statistical analysis of data, derived the formulae in c.f.s. units (except d) as follows:

$$P = 2.68\, Q^{\frac{1}{2}} \qquad (13)$$

$$S = 0.00209 \frac{d^{0.86}}{Q^{0.21}} \qquad (14)$$

$$\frac{R}{P} = \frac{S^{\frac{1}{4}}}{6.25\, d} \qquad (15)$$

where d is the weighted mean diameter of sediment in mm.

Both the silt factor f of Lacey and the weighted mean diameter d of Bose, define the *size* of the sediment transported on or near the bed in a stable channel, but not the sediment *charge* or the *rate* at which the sediment is transported. It can be anticipated, therefore, that these formulae are applicable to canals that carry more or less the rate of sediment[2] like the canals in India and Pakistan from which the formulae were derived.

Fargue's laws for regulation of rivers

The analytical treatment given above does not indicate the lines along which the design of river training might follow. It has been pointed out before that the alignment of river is to be chosen in accordance with the final developed meandering form. The analytical treatment, being itself in the initial stage of development, has not yet been able to cover this aspect of the problem,—i.e., the form or curves of the river course. Great importance is attached to this problem for navigable rivers, as the curvature of the river course greatly influences the depth of channel for navigation.

Up to now, designs of the alignment of river course still follow the six laws formulated by Fargue when he regulated the river Garonne in the latter part of the 19th century. These laws are by no means quantitative, yet they provide the means of formulating a set of dimensions from the existing conditions of the river itself; and from the later experiments of reproducible meandering pattern, this is also a logical and sound practice. Some attempts have been made to supplement Fargue's laws with empirical formulae, like H. C. Ripley's formula to express the relation of widths and depths to the curvature of channels, but they were not so successful. Further study in models as well as natural rivers will surely reveal the theoretical as well as practical relations between different factors of river flow. Fargue's laws are quoted below:[3]

(*a*) *Law of Deviation*

The deepest and shallowest points in the channel are below the vertex and the end of curves respectively. On the Garonne, the average of this down-stream displacement is about ¼ length of the curve but is less than that on the shallower crossings.

(*b*) *Law of Greatest Depth*

The point of maximum depth is deeper as the curvature at the vertex of the curve is sharper.

1. Lacey, G., *Regime Flow in Incoherent Alluvium*, Publication No. 20, Central Board of Irrigation (India), 1939: "Stable Channels in Alluvium" in *Proceedings of the Institution of Civil Engineers*, London, vol. 229 (1929-30), paper No. 4736.

2. The sediment content of the irrigation canals in Punjab is not high. According to F. F. Haigh (*Minutes of Proceedings of the Punjab Engineering Congress, 1938*, p. 71), the Maximum sediment content expressed in parts per thousand by volume, during monsoon period in 1935-1937 for the following canals are:

Upper Jehlum Canal at	Jaba	0.12
„ „ „ „ „	Kasba	0.029
„ „ „ „ „	Chak Sikander	0.091
„ „ „ „ „	Dumbi Wala	0.146
Lower Chenab Canal at	Khanki	0.164
Upper Bai Doab Canal at	Madhopur	0.232
Western Jumna Canal at	Dadupur	0.131

3. Fargue, M., *op. cit.*

(*c*) *Law of Trace*

In the interest of both average and maximum depths, the curves should be made neither too short nor too long. For the Garonne, the length is preferably about 1.3 km instead of perhaps 50 per cent greater or lesser than that.

(*d*) *Law of Angle*

To equal length of curves, the average depth of the pool is greater as the central angle sustained by the curve is the larger.

(*e*) *Law of Continuity*

The longitudinal channel profile shows gradual variation only when the curvature changes gradually. Abrupt modification of depth accompanies rapid variation of curvature.

(*f*) *Law of the Slope of the Bed*

If curvature varies continuously, an increasing radius of curvature marks a reducing depth and an increasing degree of curvature is accompanied by a deepening.

Fargue's laws are generally accepted by most engineers in river regulation. When the improvement works require not only a given depth but also that this depth should be reached over a stated and fairly wide width (which was found to be so on accentuated curves), the application of Fargue's laws is not advisable.

Some conclusions[1] arrived at by the XVth International Congress of Navigation for the second question of Inland Navigation—Regulation and canalization works of rivers and streams —are useful in connexion with the application of Fargue's laws and are quoted below:

(*a*) In seeking to maintain suitable depths, physical and hydraulic changes must be kept within the limits imposed by hydrographic conditions and by circumstances relative to the utilization of streams for other purposes in the public interest.

(*b*) As an indispensable guide for the attainment of results as near as possible to truth, one should use design formulae for the computation of erosion and for the movement of sediment, whose coefficients are deducible from the stream under study.

(*c*) Before starting the regularization, it is desirable to observe the movement of transported material both on bed and in suspension, to note carefully the changes in the depth of bars, in relation to changes of water level. This is particularly important with sandy bed streams and high velocity of current during flood flow stages. During the work and after completion, the observation in question should be continued with the same motive. At the same time, longitudinal and transverse profiles should be taken especially during characteristic stages of low water, medium water and after floods.

(*d*) It is a well-known fact that the contraction of a stream causes a reduction in the slope of its bed. This has been confirmed almost universally in streams that have been already regulated. Such a change of slope may be calculated in advance, at best approximately, taking and considering all the data which the natural stream suggests.

(*e*) From reports submitted a confirmation of the conclusions of Fargue may be deduced if they are adopted to the characteristics of the stream and if all known facts of observation and experience are considered.

Based on Fargue's law, a standard alignment as suggested by Shaffernak[2] is shown in figure 8. It may be pointed out that, when deciding an alignment in practice, a suitable "streamline" alignment is often considered more appropriate than one strictly following any fixed rule.

Hydraulic geometry of stream channels

Leopold and Maddock Jr.[3] examined the characteristics of channel shape of some twenty rivers in the United States and found out that, up to the bankful stage, the width, depth, velocity and sediment load of river cross-section possessed definite relations with the discharge. The same applied also to stable (regime) irrigation canals where neither scour nor aggradation occurred. The analogy demonstrated that the average river channel system tended to develop in a way to produce an approximate equilibrium between the channel and the discharge and sediment it must transport. This approximate equilibrium, or hydraulic geometry, appeared to exist even in ungraded tributaries in head-water region of a river system.

For all the cross-sections of the twenty rivers investigated, Leopold and Maddock Jr. found the following functional relations:

$$B = \alpha_1 Q^{\phi_1} \qquad (16)$$

$$D = \alpha_2 Q^{\phi_2} \qquad (17)$$

and

$$V = \alpha_3 Q^{\phi_3} \qquad (18)$$

1. *Report of the Proceedings of the XVth International Congress of Navigation*, Venice, 1931.

2. Shaffernak, F., *op. cit.*

3. Leopold, L. B. and Maddoc, Jr., T., *The Hydraulic Geometry of Stream Channels and some Physiographic Implications*, US Geological Survey Professional Paper 252. Washington D. C. 1953.

where Q = discharge through the cross-section; B = water surface width; D = mean depth; V = mean velocity; α_1, α_2, α_3 = three different coefficients; and ϕ_1, ϕ_2, ϕ_3 = three different exponents.

Since $Q = B \cdot D \cdot V = \alpha_1 Q^{\phi_1} . \alpha_2 Q^{\phi_2} . \alpha_3 Q^{\phi_3}$.

Therefore $\alpha_1 . \alpha_2 . \alpha_3 = 1$

$$\phi_1 + \phi_2 + \phi_3 = 1$$

From an analysis of data of twenty rivers (or cross-sections) Leopold and Maddock Jr. found the average value of ϕ_1, ϕ_2 and ϕ_3 to be 0.26, 0.40 and 0.34 respectively.

Leopold and Maddock Jr. also found that the width, depth and velocity of a river increased towards downstream. The functional relations stated above also held true downstream, with a different set of ϕ_1, ϕ_2 and ϕ_3, if discharges of equal frequency, that is, discharges equalled or exceeded the same per cent of time within a year, were used at different points along a river.

They explained that this increase of velocity downstream resulted from the fact that the increase in depth over-compensated for the decrease in slope. This tendency for velocity to increase in the downstream reaches of natural channels existed on most streams despite the decreasing size of bed particles downstream. From this, they concluded that the mean velocity in a given reach was not merely a function of the size of particles which must be transported but was governed by a more complex interaction among several factors. Average values of ϕ_1, ϕ_2 and ϕ_3 for the downstream reaches of the 20 rivers studied by Leopold and Maddock Jr. came out to be 0.5, 0.4 and 0.1 respectively.

From an analysis of data, Leopold and Maddock Jr. concluded that the relation of suspended sediment[1] to discharge at a given river connection might be approximated by a straight line plotted on logarithmic paper. Such a line is generally defined by the expression

$$\text{Sediment load (c)} = CQ^{j} \qquad (19)$$

In the data analyzed, there is a tendency for the suspended sediment at a station to increase faster than discharge, and the value of j typically lies in the range 2.0 to 3.0. It was found that for any given rate of increase of width with discharge—that is for any given value of ϕ_1, the value of j increased with an increase in the ratio ϕ_3/ϕ_2. For average conditions both at a station and downstream the corresponding values of j were found to be 2.3 and 0.8 respectively.

At a given discharge, an increase in width at constant velocity is accompanied by a decrease in suspended load; conversely, at constant discharge and velocity, an increase in width is accompanied by an increase in bed load. It can be said that the scour and fill of the bed of the main stem of an alluvial river during flood appears to be an adjustment of channel shape in response to a varying sediment load.

1. Not including the bed load.

Model tests on river training

In the design of river training, there are no standard procedures to be adopted as in the case of the design of a dam or barrage. However, with the development of the science of experimental hydraulics, the method of hydraulic model testing has been found most useful in solving many intricate problems in river training.

The purpose of model tests is to determine the effect of any contemplated river training scheme on the silting or scouring of the river bed and banks by trying out the scheme on a scale model. Basically, the model must be similar to the prototype in respect to sediment movement. As the law governing the movement of sediment in rivers is not yet very well established, the selection of model sediment and the operation of the model so that sediment movement will be similar to the prototype is also done by the method of trial and error. There are two pre-requisites to such an approach: firstly, a thorough knowledge of the characteristics of the prototype based on the collection and study of hydraulic and hydrological data including sediment in the natural stream; and secondly, experience in the field of movable bed models, in particular when the model is distorted.[2]

In conducting the model test, two or more prototype bed surveys of past dates are selected and the model bed is moulded to conform to one of the earlier surveys. The model is then tested with certain discharges corresponding to prototype discharges. The time (over a certain period from low water to high water and then back to low water) is varied by cut and try till the model bed exhibits changes similar to those that have occurred in the prototype. In some instances even the model discharge or slope has to be arbitrarily changed with a view to obtaining similar bed movement. Such a procedure is called verification of the model and is considered absolutely necessary. It calls for an intimate knowledge of the prototype as well as experience with this type of model study.

After a satisfactory verification of a river model is obtained, any contemplated training scheme is tested in the model and the results of changes of river bed and banks are carefully observed. If the model accurately reproduces changes of river bed which are known to have occurred in the bed of the prototype, it

2. If the velocity to be obtained on a model is too low to be capable of movin a light weight bed material selected for the model, horizontal and vertical scales are chosen different giving the model a vertical distortion (greater depth).

can be relied upon in predicting changes of a similar nature which can be expected to occur in the future. However, since the verification is achieved on the basis of an adjusted simulation of recorded prototype phenomena, the model cannot be expected to respond accurately to conditions which involve a drastic departure from those involved in its verification. In the final analysis, the validity of the results of a movable-bed model study and the interpretation of its results are largely dependent on sound judgment and reasoning.

Notwithstanding the limitations mentioned above, model tests have been proved to be indispensable in planning river training. With the progress made by the various laboratories, it is possible not only to determine the river bed conditions and training works in a model qualitatively, but also to some degree quantatively, provided accurate and adequate data on the prototype were available. Examples of such tests are the experiments carried out on models of the Rhine river. This is confined to rivers where bed load is the governing factor. As far as sediment flow in suspension is concerned, the theory itself has not yet been clarified enough to enable hydraulic similitude to be obtained in models.

Model tests are usually conducted for individual projects and for specific purposes. Systematic investigations on theory of river training by means of models had not been done until Friedkin[1] conducted systematic experiments at the US Waterways Experiment Station regarding the changes brought about in the channel of a meandering river by stabilization of the caving banks. Two types of model tests were conducted: firstly, the caving banks of laboratory rivers, which consisted simply of a series of nearly uniform bends which developed from a straight or slightly sinuous channel, were stabilized; secondly, the caving banks of a small-scale river, which had initially the alignment of a stretch of the Lower Mississippi river, were stabilized.

These tests showed that the result of stabilization in a model of the eroding banks of a small-scale meandering river, which consisted of a series of nearly uniform bends, was the deepening of the channel along the talweg. Generally, the depth of channel along the talweg increased more in the bends than over the crossings. The low-water depths over the crossings, however, remained about the same or increased slightly. With bends of moderate curvature (sinuosity of 1.30), the stages for maximum flows were definitely lowered after the banks were stabilized, while with bends which had a fairly high degree of curvature or those that had a low degree of curvature, the stages for maximum flows did not change materially.

1. Friedkin, J. F., *op. cit.*

Stabilization of rapidly caving banks of small-scale rivers which had initially the alignment of a section of the Mississippi river resulted in deepening of the talweg along the stabilized banks which were under attack. Depth of low flows over the crossings increased. The stage for maximum flow did not change, or the channel maintained its capacity; whereas in similar tests, in which the banks were not stabilized, the stages of maximum flow became higher.

Low water and high water training

Low water training aims at the increase of water depth for purposes of navigation. This is usually done by a further contraction of river bed by means of a system of low groynes up to low water level. The groynes may be a separate system or an extension from the groynes constructed for mean water training. If a secondary meandering is developed within the mean water bed, the low water training works may also be designed for the purpose of training the river bed, particularly at the crossings, besides the general function of increasing the water depth.

With the low water groynes in place, the cross-section of the river below the mean water level usually takes the form of a composite profile. The alignment of the low water channel follows that of the mean water with a further shifting toward the concave bank. A general alignment as suggested by Schaffernak is given in figure 9.

In high water training for flood control, the important problems are the distance of the embankment from the mean water course and the alignment of the embankments. The distance of embankment from the mean water bed controls the area available for flood discharge, which, together with the height of embankment, is to be decided by a consideration of flood discharge. The alignment of embankment is important with respect to both flood flow and low water training. Whirlpools, which usually develop at places downstream of a convex curve of embankment and thus reduce the effective cross-sectional area for flood discharge,[2] should be avoided by giving a less winding course than the mean water channel. The embankments should be so aligned, if not prevented by local obstructions, that the axis of flood flow should coincide more or less with the mean water channel and should not cross it at the convex bank. The embankments are usually placed nearer to the mean water channel at the concave bank of the latter. A general lay-out is also shown in figure 9.

2. Wittman, H. and Boess, P., *Wasser und Geschiebebewung in gekruemmten Fluessen*, Julius Springer, Berlin, 1953; see also Wittman, H., "Untersuchungen am Rehein usw" in *Der Rhein, Ausbau, Verkehr, Verwaltung*, (published by Rhein Verlagsgessellschaft Duisburg, 1951), p. 81.

Training of rivers other than the meandering type

It should be noted here that in most cases, river training is applied to rivers of meandering type for the purpose of improvement for navigation. A meandering river in its natural stage is by no means composed of reversed curves all through its course. There are reaches where the course as a whole is fairly straight and the flow is usually divided into two or several channels by shoals or bars in between. Perhaps such a reach is the upper limit of the state of meandering, and how it is formed is still a subject needing further investigation. However, the artificial training of a river usually requires the change of such a reach into more or less a meandering section with a single channel of flow, otherwise the river would probably not attain its greatest depth at shoals. This further proves that meandering is the final and therefore the equilibrium stage of river development and only by following the rules of meandering can a river reach a condition of stability, which is the aim of river training.

Rivers other than the meandering type, i.e. the aggrading and degrading type, cannot therefore be trained to attain the stage of stability unless other measures are taken jointly with training works. For instance, any training work built on a river of a degrading type would eventually be destroyed by the scouring of the bed, which is a characteristic of this type of river, unless some measure is taken to correct the slope of the river, say by building of weirs or by canalization, although canalization is not limited to rivers of the degrading type, as it is also warranted, when necessary, for navigation. For instance, the Rhone in France and the Upper Mississippi in the United States of America were both improved first by regulation, but later by canalization.

Aggrading rivers may be benefitted by bank stabilization to a certain degree, but unless this stabilization of banks or the reduction of sediment charge as a result of stabilization enables the river to cope with the incoming sediment load, there may be a tendency for the training works to be either buried under the deposition or destroyed by the current in the course of years. Therefore, training, to be really effective, must be accompanied by soil conservation measures in the uplands.

Short stretches of an aggrading river may however be trained to answer certain purposes such as contracting a channel for a bridge, etc., but not without limit. The river slope in this short reach can be changed to a certain extent, provided its effect upon the downstream and upstream sections is not too drastic.

METHODS OF RIVER TRAINING

Selection of alignment and determination of normal cross-section

In the planning of river training, the first step is to determine the alignment of the mean water course and the dimensions and form of the normal cross-section. The opportunity and the necessity of changing the alignment of a complete length of a river, or of designing a completely new water course for the whole length of a river, seldom occurs. Training is usually carried out where the alignment of a section of the river is rather abnormal, such as the splitting of the channel into branches, the development of sharp bends, and the formation of wide and shallow crossings. It is also carried out for some specific purposes such as the leading of flow through a bridge or the diversion of flow to avoid bank erosion.

The selection of alignment is usually governed by the following considerations: (*a*) the necessary number of training structures involved such as training walls and groynes and the volume of river bed material to be dredged or expected to be washed away by river training so that a minimum amount of material and labour are required; (*b*) the principles given by Fargue; and (*c*) a good alignment of the river itself in the vicinity.

If it is intended to maintain the river in a single channel at places where it branches into many arms, the branch carrying the major portion of the discharge is usually chosen.

The size and the form of the cross-section of a river is governed by the discharge, velocity, river bed material, rate of sediment flow and many other factors which vary not only from river to river, but also from place to place on the same river. Up to now, no definite formulae have been established to enable exact computations to be made. The general practice is to adopt the good or "normal cross-section" built up by the river itself. Investigation is made on the size and form of cross-sections at the *crossings* of a good stretch of the river, where the channel is uniform and shows no sign of aggradation or degradation. After such a normal cross-section is chosen, it is then checked and modified according to the following calculation:

(*a*) Calculate the relation between water level and discharge of the new section, based on which investigation is to be made to ascertain the suitability of water level and velocity in respect of navigation and flood control (height of dikes).

(*b*) Calculate the annual bed load run-off of the new section; compare the annual bed load run-off with that of the normal section of the river upstream of the training reach.

(*c*) Modify the cross-section and repeat the calculation mentioned above until water level and velocity correspond with the desired values and that the annual bed load run-off is equal to that of the normal section of the river.

Lay-out of river training works

Ordinary rivers are usually too wide and therefore too shallow. This is remedied by the so-called river contraction. Bank caving is the main contributory cause of river deterioration, and the stabilization of banks, usually by revetment, is an important part of the work in river training. Bank revetment is usually provided where little or no correction of curvature, or contraction of channel is needed. Otherwise, training walls and groynes are used for contraction or correction of curvature. Where the river has developed a sharp bend and an extremely deep pool, sills are usually constructed to correct the depth, thus stopping the further development of sharp bends. In European practice, sills are provided at bends, usually extending from a revetted bank or from a training wall, to prevent the river bed from further scouring as a result of the improvement works.

Closing of river arms is usually done by building of closing dikes across the upper end of the channel with due regard to the alignment of the bank line.

Training of a short stretch of a river for guiding it through a bridge or an irrigation head works may be accomplished by construction of training walls or groynes or both.

When developing training works for a relatively low reach of river, the work should begin at the upstream and extend progressively downstream. This is an important step because any changes occurring upstream will eventually affect the regime downstream.

It is usually beneficial to start river training works after verification of the plan by means of model tests. Flexibility of construction is desirable so that correction can be made easily, whenever any adverse effect results.

River contraction

River contraction is usually accomplished by constructing revetment or training wall on the concave bank and groynes on the convex bank, with or without sills connecting the revetted bank or training wall. At crossings, groynes are used on both sides, and are usually built in pairs, so that the centre line of each pair of groynes from both banks will intersect at the centre line of the talweg.

For the purpose of improvement of navigable waterways, river contraction works may not extend much above the median water level of the river; and for minor bed regulation, the height of works may be limited to low water only, as otherwise they would disturb the regime of normal flow.

There are cases where groynes are extended to a higher level such as in the Missouri. The construction of permeable groynes on the Missouri was in fact for low water regulation only, although the groynes were extended to a height just approximately below bankfull stage. The reasons for the adoption of this course were that (*a*) high groynes intercept a greater amount of silt and create deposition faster than low ones, as the total amount of silt carried at higher stages is greater than that at low stages; (*b*) low groynes are submerged at higher stages and tow boats and barges are apt to run over them and be punched in the bottom, in spite of efforts to keep the outer ends well marked; and (*c*) higher flow over low groynes produces turbulence and powerful scour just below them, thereby preventing them from creating fill. However, there are several factors that limit the height of these pile structures. Firstly, the cost of long piles would be unnecessarily great if the groyne is built unduly high. Secondly, difficulty may be encountered in securing sufficiently long piles, with reasonable penetration (normally 20 ft, increased to 30 ft at the outer ends), so that the groynes remain weak against overturning or destruction, when attacked by drift or ice. Thirdly, experience has shown that low groynes of this type (below mean stage) will secure fill behind them and become effective during low water seasons. Fourthly, the range of stage which might create strong jumps over such groynes is limited.

In case the contraction is for guiding the flow through a bridge opening, weir or regulator, when the flood flow as well as the low water flow of the river must be dealt with, the contraction works are usually built with a free board above the maximum flood. The scouring that will result after contraction is also estimated for the maximum flood flow. The detailed design of such a contracted section will be described in section *River training works in the region.*

Groynes

Groynes are structures built transverse to the river flow, extending from the bank to the limit of normal lines. Many different names are given to this type of structure, the most common being "spurs", "spur dikes", "transverse dikes", "jetties", etc.

Groynes are usually constructed in series, as the purpose of using groynes is to accumulate silt in the spaces in between them thereby forming a permanent bank line in accordance with the design. A single groyne is neither strong enough to deflect the current nor so effective as to deposit silt upstream and downstream of it. Owing to the disturbance caused to the natural flow, a single groyne may easily be more harmful than useful. Groynes may be built behind training walls which are far away from the original banks, in order to strengthen the structure and to prevent the formation of a channel of flow whenever there is a breach through or overtopping of the training wall. This is very much like the "keh-ti" or "cross-dikes" used on the Yellow River, where two parallel embankments were constructed on each side of the river, the one nearer the river being for the purpose of training and contracting the channel and the other, further away, for protection from flood, with "cross dikes" connecting them at certain intervals to prevent the formation of new channels.

Groynes may be either perpendicular to the bank lines or to the talweg, or at an angle to this perpendicular facing slightly upstream or downstream. Sometimes the angles between the centre line of the groynes and the talweg may change continuously starting from the first one in a series to the last one according to the curvature of the bank line. Generally speaking, the head of a groyne causes disturbance of flow at the nose and heavy scouring usually results downstream due to eddy formation in the direction approximately perpendicular to the groyne. Groynes pointing downstream would bring the scour hole closer to the bank than in the two other cases, viz. where the groyne is perpendicular to the bank or is pointing upstream. Therefore groynes pointing downstream endanger the bank unless the next groyne is near enough to be effective in eliminating such danger. The present practice, therefore, tends to plan the construction of groynes either perpendicular to the bank or inclining toward the upstream, the former being used mostly on convex banks. The inclination varies from 8 degrees to as much as 60 degrees with the upstream bank or in other words, the centre line of the groyne makes an angle of 10 to 30 degrees with the line perpendicular to the bank or talweg. The length of groynes depends upon the distance between the original river bank line and the designed normal line of the trained channel. Too long a groyne is not desirable on rivers with easily erodible material (as compared with the strength of the flood current). In this case the best way is to start with a shorter length and to extend the groynes after space between them has been silted up. Shorter and temporary groynes constructed in between the long ones may be necessary to induce silt deposition.

As regards the distance between adjacent groynes, the general practice is to make it in a certain proportion of their lengths, varying it with the width of the river. Groynes are usually spaced further apart (with respect to their lengths) in a wide river than in a narrow one if the discharges of the two are more or less the same. The location of the groynes, that is whether they are at a concave or convex bank, or at a crossing, also affects their spacing. Larger spacing can be planned for convex banks, and smaller for concave banks, with spacing at the crossings in-between the two. Types of construction also affect the proper spacing. Usually, permeable type of groynes may be spaced further apart than the solid type. A spacing of two to two and half times the length of groynes at convex banks, and equal to the length at concave banks is the general practice. Groynes built behind training walls can be spaced further apart. Sometimes, engineeers may space the groynes too far apart in order to save the cost of construction, or with a view to inserting more groynes in-between them at a later date, with the result that either the river will be more disturbed and the groynes may be out flanked, or their heavy maintenance cost may exceed the saving attempted.

With their lengths and spacing fixed, other dimensions of groynes will depend on the type of construction and the extent of protection required. Any material that would resist both the water and the current might be, and has been, used for construction of groynes. There are two main types of groynes, the solid type and the permeable type; and there are also two heights of protection, the high water and the low water. Dimensions of groynes should be designed according to (*a*) the material used; for instance, an earth groyne cannot be overtopped and must be safely above the highest possible water level; (*b*) the force encountered; for example, the foundation of a pile type groyne must be protected by a sort of mattress etc., otherwise the scouring around the piles would upset a part or the whole of the structure; and (*c*) the angle of repose that the material may asssume when dumped or placed in the river; for instance, rock could have steeper side slopes than earth. Also where the attacking force is the greatest, the section should be the strongest, for instance, the head of a groyne is usually strengthened by either a flat slope protected with a strong armour on its surface and specially at its toe, or by a strong head of T, L, or hooked form. The longitudinal slope of the groyne from the bank to its head usually depends on the height of the groyne. Groynes must be well protected at their bank ends in order to avoid out flanking by the river flow along the banks.

Permeable vs. solid groynes

Solid groynes may be built of earth armoured with resistant material, of rubble mound, of rubble and fascine mattress, or cribs filled with stone, and many other types. They are designed to deflect the current to a course further away from the bank. It is expected that owing to formation of back rollers or anti-erosion eddy—or pressure eddy as called on the Mississippi—within the area between the groynes, sediment carried by the river will be deposited and eventually will form a permanent bar or higher ground along the river. The deposition is, however, usually slow especially in the case of high flood protection groynes in rivers with tortuous low water courses. The river will simply adhere to a portion of the upstream toe of the groyne, passing round the head, if the head is of a hockey stick shape or round and thence flowing obliquely towards the bank downstream. This condition may prevail for years, with the constant danger of a breach through the groyne. This is the condition of what is called an attracting spur in India, which has a tendency to attract the current toward the bank instead of deflecting it away. The head of the groyne, which is heavily attacked by the current, would require constant attention and repairs.

A permeable groyne of the hurdle or pile cluster type has the advantage of slowing down the current instead of deflecting it, thus hastening the deposition of sediment and building up of the high ground and the bank lines. This is especially effective on rivers carrying a considerable amount of suspended sediment. Permeable groynes have been used in India and Pakistan as a temporary measure to meet exigencies with success. But permeable groynes have their draw-backs; because (*a*) they are not as strong as the solid types in resisting the forces exerted by floating debris or ice; (*b*) they are a nuisance to navigation during high water period if built low, and the cost of long piles may be prohibitive if built high; (*c*) the best type of permeable construction is obtained by using piles, but this would not be feasible where the river bed is of gravel and boulders; they will not resist the flow very well unless they be of the tetrahedral type such as those used in California and Japan, where slant posts connected to base beams are weighted down by concrete blocks or stone in wire crates.

The selection of a solid or permeable type of groyne is therefore entirely dependent on local conditions with perhaps a preference for the permeable type if the river carries a large portion of suspended load. Where transportation of sand and gravel are predominant in quantity over suspended silt, solid groynes are preferable.

Training walls and bank revetment

Various names have been given to the type of structure built parallel to the current and connected to banks, such as "longitudinal dikes", "parallel dikes", "jetties", "guide banks", etc. As these structures directly train and guide the flow of a river, the term "training walls" is used throughout this text. Bank revetment is in fact a sort of training wall resting on the natural river bank, but as its construction may differ from an ordinary training wall and its purposes are manifold, it will be treated more fully in the next chapter.

Training walls are usually built of the solid type, either of rubble mound on a natural or artificial foundation or of fascine mattress with rubble stone; or with a core of gravel and sand faced with rubble or fascines. They should be connected to the river bank at their commencement, and should join the normal lines in suitable curves. They may stop at a certain point along normal line or may be connected to the banks at their lower ends also. Training walls are generally constructed along concave banks where the natural banks are too far away from the normal lines. For low water regulation, they are of such a height that their crest is a little above the low water level. In Europe, it is the practice to have the training wall at a level a little lower than the water level from the bank to the point of inflexion of the curve connected to the normal line, and its height is gradually increased towards the bend of the normal line, being highest at the apex. The height is then gradually reduced to a minimum at the next point of inflexion.

Jetties built from a river mouth to the sea, in order to concentrate the flow and the tide so as to eliminate the formation of a bar at the river mouth, or to deepen the channel for navigation, are in the true sense also training walls, but they are made of heavier construction material and are higher than the training walls in river reaches, to enable them to resist the wave action of tides and to guide high tidal flows. Their sections resemble those of breakwaters.

The construction of permeable type of training walls along caving banks is practised with a view to building up foreshores to the original bank line. This is in reality a bank protection scheme, but if worked well, it has the result of river training. Permeable and light type of construction made of bamboo stakes and waling pieces, woven with bamboo sticks to form a curtain of bamboos, has been used in Burma for training streams to flow in confined channels and to build up bank lines by intercepting the silt by means of these fence works. This is called "river training without embankments" in Burma.

Groynes vs. training walls

As training walls are constructed in a continuous line parallel and usually not too far from the banks while groynes are usually spaced further apart than their length, it is self-evident that groynes are cheaper in construction cost than training walls. Moreover, training walls are exposed to attack of current, and are also subject to water pressure whenever there is a difference of water levels in front and back of the wall, while groynes require protection only along their heads, and, as the difference of water level between the upstream and downstream faces is generally not so great, a smaller section is required for groynes than for training walls; it is again evident that groynes are cheaper to build. Another important point is the flexibility possessed by groynes. River training works are even now more of an art than a science and therefore, there is always the chance of the revision of any plan after it has been carried out. Groynes offer ease of revision by extending or shortening a portion of the structure to the new revised limit without altering the remaining portion. Training walls cannot be corrected, if it is subsequently found that they narrow the river too much. When the training walls are found to be too far away from the normal lines, groynes are used for extension, thus involving great expense.

But groynes have their inherent defects, in as much as they disturb the current and are a menace to navigation, when the river rises just higher than their heads. As Salberg[1] has put it: "A spur or groyne was usually anathema to the ordinary river engineer." This is especially true when groynes are situated on concave banks where erosion is active. To avoid too much disturbance and to ensure the safety of the training works, it is a sound engineering practice to build training walls on the concave banks instead of groynes. Again, the head of a solid groyne is a point creating eddies, and is subject to severe scour. The constant attendance and repairs required are very disadvantageous to groynes.

Sills and closing dikes

Sills, sometimes called "submerged sills" or "submerged dikes", are used to correct the excessive depth of a channel where the curvature is greatest and erosion is most active. In Europe, it is the general practice to plan sills at the foot of a training wall, wherever the depth of the channel exceeds the designed value so as to protect it from undue scour. In fact, it is an extension of the training wall, just like aprons for bank revetment.

1. Salberg: Discussion on "Training works constructed in the Rupnarain River in Bengal", Institute of Civil Engineers, Maritime Paper No. 3 (1946).

Sills are usually built of rubble mound or rubble and fascine mattress, across the deep pools of the river bed with their crests located at or below the designed bed level. They must be spaced close enough to ensure their functioning properly to stop the scour.

Closing dikes, to cut off the flow of a river branch, are in fact training walls for leading water to a single channel around the bar. They are usually placed at the upper end of a branch channel, and the channel may silt up by backwater during floods. The dike, if of the solid type, must be strong enough to resist the pressure difference created during high water. The dike can also be of the permeable type, just to retard the flow and to hasten sediment deposition. In case the flow through the branch is swift, it may be necessary to build more than one permeable dike below its offtake. In tidal rivers, dikes may be necessary at both ends.

Cut-offs

When the meandering of a river develops to extreme conditions in the shape of horse-shoe bends, the land between them may gradually dwindle down to a narrow neck which may easily be cut across by natural flow during floods, or by excessive erosion. These natural cut-offs often result in violent changes in river regime, steepening the slope, inducing erosion in the stretch above the cut-off, and lowering the water level upstream. At the same time, owing to the reduction of channel storage above the cut-off, the in-rushing of flood may be a menace to the lower stretch. Also there is the chance of deterioration of the river channel downstream due to the flow of excessive eroded material from upstream. Moreover, since a cut-off can change the river regime only temporarily, there will be a tendency for the river to adjust itself and find a new equilibrium by lengthening itself in some other curves. This has been proved by the survey made on the Lower Mississippi, which showed that this river, although shifting its course considerably, with occasional natural cut-offs, did not change its total length to any appreciable extent. This was also the reason why engineers, working on or investigating the flood problem of the Mississippi in the former days, firmly condemned the adoption of cut-offs for shortening the travel route of floods and instead, recommended the protection of banks in order to avoid any natural cut-offs.

The problem of cut-offs on the Mississippi was solved, however, by model tests and by the use of pilot cut, which is an artificial method of chute development on natural rivers, and did not affect the river regime as violently as expected. The pilot cut usually carried 8 to 10 per cent of the discharge at the beginning and gradually enlarged its capacity in the course of two to three years. Large scale cut-offs were carried out on the Lower Mississippi. Sixteen cut-offs between Memphis, Tenn., and Boston Range, La., aggregating a total length of 168,205 ft, shortened the river by 151.8 mi net, from an original length of 452 mi. The effect on the flood water levels was a lowering at Vicksburg gauge amounting to 7-8 ft for five cut-offs below Vicksburg, Missi., and 12-13 ft at Arkansas City, Ark. gauge[1] for twelve cut-offs below that point.

The cut-offs mentioned above were mainly for the reduction of flood heights and flood periods. As to navigation, the benefit lays in the shortening of the distance of travel and the ease of manoeuvering along bends; but owing to increase in velocity of the current, loss may sometimes be greater than the benefit. In tidal rivers, however, the condition is different, as there is a two way flow instead of one way, specially when the incrsease of tidal range upstream is very essential for navigation. This has been shown by cut-offs on the Hai River in China in which no pilot cut was used. Similarly, no pilot cut was used in cut-offs in the Lower Rhine in Holland, where suitable dredging equipment is available to operat cut-offs in one step in order to avoid many irregular situations that may arise from pilot cuts with respect to navigation as well as flood control.

Dredging

Dredging is an essential part of the work of river training, not only for cut-offs as mentioned above, but also for other part of river works, where adjustment of configuration of the channel is required and where currents are too slow to allow practical result from training alone. Dredging, if properly carried out, will help the river to assume the new regime imposed on it at an earlier date and save the cost of maintenance as well as the inconvenience caused by changing conditions for navigation. Even after a river has been trained for navigation, dredging may be required occasionally, when the variation of flow or of sediment charge may produce certain adverse effects on the channel. Whenever dredging is attempted, the possibility of re-silting should be carefully studied in advance.

RIVER TRAINING WORKS IN THE REGION

Guide banks (India)

Indian rivers in the plains are usually wide and shallow and are liable to shift rapidly owing to caving and shoaling. Alluvium deposits are usually thick and, therefore, extraordinary scour causing deep holes is not uncommon on Indian rivers. These conditions taxed the ingenuity of India's bridge engineers to find a solution for their rail and road bridges, so that these bridges may (1) not be outflanked by the changing river banks, and (2) not be so long as to increase the cost of the superstructure and yet (3) be not so short as to require unduly deep foundations. The introduction by Bell of the so-called Bell's Bunds, which were further developed by Spring,[2] and were generally known as guide banks, were certainly a marvellous piece of engineering contribution, although much room was left for perfection. The bunds usually consisted of two heavily built embankments or guide banks in the river in the form of a bell mouth, the distance between the banks being only a fraction of the width of the river—occasionally only one was required if a high bank could be used on one side. The guide banks served as bridge abutments. The remaining portion of the river between the heavy embankment to the bank was crossed by ordinary embankments.

The theory of the use of guide banks is explained as follows. In nature a wide river often passes through a gorge with reduced cross-section and a much reduced width, without creating abnormal velocities. It was considered, therefore, that a gorge of this kind could be built artificially by means of a pair of strong banks, spaced sufficiently apart to pass the entire flood flow with no more than its natural slope, but with a greatly improved section. The criteria for narrowing the flow passage are two: on the one hand, there should be sufficient depth of alluvium so that the river may scour down to provide enough cross sectional area for flood flow, and on the other hand, the increased cost of foundations together with the reduced cost of superstructure should be a minimum. The design of such guide banks involves the following considerations: (*a*) the ultimate width to which an alluvial river can be contracted; (*b*) the length of the guide banks; (*c*) the plan for the banks that will lead the flow efficiently through the bridge and at the same time prevent them from breaching as a result of embayment caused by the banks both upstream and downstream or owing to extraordinary scour caused by the formation of swirls or eddies; (*d*) the maximum depth of possible scour both along the guide banks and the bridge piers; and (*e*) the method of protecting the structure against scour. The above considerations are discussed below in greater detail:

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1. Matthes, G. H., "Mississippi River Cut-offs," in *Transactions of the American Society of Civil Engineers*, vol. 113, 1948, p. 1.
2. Spring, Francis J. E., *River Training and Control on the Guide Bank System*, technical paper No. 153, reprint 1935, Manager of Publications, Delhi, India.

(*a*) The width of the artificial narrow gorge can be easily calculated by Lacey's regime formula. Approximately, the width $B = 2.8\sqrt{Q}$, provided the alluvium can be scoured down sufficiently to suit this width, Q being the estimated maximum discharge in cu ft/sec and B in ft.

(*b*) The length of the upstream guide banks is made about equal to, or up to a length longer than the length of the bridge. In specially wide "khadirs" or low lands of river meanders, the possibility of the river going round a guide bank into the still water area at its back and eroding the main approach embankment may involve the use of very long guide banks. However, too long guide banks are usually avoided by the addition of groynes.

The length of the downstream part of the guide bank may be a tenth to a fifth of the length of the bridge, according to the judgment of the engineer regarding the activity of swirls or disturbances likely to be caused by the spreading out of the flow on leaving the bridge. If there is any swirl, it must be kept sufficiently away from the bank to prevent it from endangering the approach embankment.

(*c*) A symmetrical lay-out of the guide banks in plan is very essential to the successful directing of the flow, as otherwise the unbalanced attack of the scouring force may give one bank an unduly heavier burden than the other. If there is a high bank on one side of the river, it can be used so as to reduce the cost, and the other bank can be modeled accordingly. Whether the two guide banks are made parallel, diverging or converging at their upper end will depend on local conditions of the river, although Bell recommended a divergent form that is the banks should be brought closer together at their upstream ends than at the bridge site, so that the area of the narrowest part near the upper end may be approximately equal to the effective area at the bridge site, excluding the obstruction of piers and the stone protection around them. Bank aprons in deep water are to be avoided as much as possible.

Both the upstream and downstream ends of banks are curved gently so as to lead the flow along smooth lines. The curve at the upper end carried well behind the bank, sustaining an angle of fully 120° to 140° and with a radius of 500 to 1,000 ft (150—300 m), depending on the probable estimated velocity that is likely to occur near it. The radius of curvature of the downstream end of the bank may be such that a main line train can runover the bank, say from 300 to 600 ft (90 to 240 m), because it is convenient to carry the stone service line over it to ensure the transportation of reserve stone during times of emergency. Table 3 given by Spring[1] is reproduced below to give some idea of the lay-out under different conditions.

Table 3

Sand standard	Probable maximum abnormal scour (ft)	Fall per mile of river in inches				
		3	6	9	12	18
		Radius of upstream curved end of guide bank in feet				
Very coarse (25% of whole retained by 16 wire-sieve)	under 20 above 20	200 250	250 310	300 375	350 440	400 500
Coarse (80% retained by 40 wire-sieves)	under 30 above 30	300 350	360 430	425 520	490 590	670 700
Medium (grade between coarse and fine)	under 40 above 40	400 450	425 550	550 650	625 750	850 850
Fine (80% passes 75 wire-sieve)	under 50 above 50	500 600	590 725	675 825	760 925	850 1,020
Very fine (20% passes 100 wire-sieves)	under 60 above 60	600 800	700 900	800 1,000	900 1,100	1,000 1,200
Radius of downstream ends of guide bank		Half the above, with the minimum radius that maintenance trains can be shunted				

1. Spring, Francis J.E., *op. cit.*, p. 157.

(*d*) Lacey[1] gives the approximate maximum depth of scour at bends as between 1.5 to 1.75 times the "regime depth" D with $D = 0.47 \left(\frac{Q}{f}\right)^{\frac{1}{3}}$ where $f = 1.76 \sqrt{d_m}$ and d_m is the weighted mean diameter of the bed material in mm. On the basis of recent studies on models and of scour data, Inglis and Joglekar[2] give the following empirical formulae:

Maximum scour downstream of a bridge

$$D_s = 1.9 \left(\frac{Q}{f}\right)^{\frac{1}{3}} \qquad (20)$$

Maximum scour round bridge piers

$$D_s = 0.95 \left(\frac{Q}{f}\right)^{\frac{1}{3}} \qquad (21)$$

Maximum scour at noses of guide banks of large radius

$$D_s = 1.3 \left(\frac{Q}{f}\right)^{\frac{1}{3}} \qquad (22)$$

in which D_s is the depth of scour in ft, Q is the maximum discharge in cu ft/sec, and f is the Lacey silt factor.

(*e*) Guide banks are usually made of material obtained locally, consisting mostly of river sand. They are provided with sufficient top width and side slopes (about 1 to 2 mostly) and protected by "one-man" stone[3] pitching, the total thickness of which is from 16 to 52 in (0.4 to 1.3m) as given by Spring. Inglis, however, gives the following empirical formula for the thickness of stone pitching required for bank protection:

$$T = 0.60\ Q^{\frac{1}{3}} \qquad (23)$$

where T is the thickness in ft and Q is the maximum discharge in cu ft/sec.

The toe of the slope is protected by the so-called "falling apron" which is in reality a pile of "one-man" stone extending from the bottom of the slope pitching to the river bed, as far as 1.5 times the maximum scour less the original river depth or equal to $2.15\left(\frac{Q}{f}\right)^{\frac{1}{3}} - R$. This assumes that extra ordinary scour may be 50 per cent greater than the maximum depth of scour given by equation (22). Whenever any erosion occurs at the outer end of the stone apron, the stone will launch from the pile to form a protected slope. Further scour will launch more stone from the pile until the maximum scour depth is reached. If the slope of the final bank be taken as S, and assuming that 25 per cent of the stone laid in the falling apron fails to settle on the slope, the total quantity of apron stone required per running foot to protect the portion of the bank face below low water level would be

$$0.08Q^{1/3} \left\{ 1.3 \left(\frac{Q}{f}\right)^{\frac{1}{3}} - R \right\} \sqrt{1 + S^2} \qquad (24)$$

where R = depth of water between maximum flood height and the original bed. If further reinforcement is required when scour occurs, another falling apron may be laid at the new low water bed formed after recession of the flow. Inglis and Joglekar[4] make the following important comments on the falling apron method of protection:

> "A falling apron provides a highly satisfactory form of bank protection where the material of the river bed or sea coast consists of sand or other incoherent material; but it is quite unsuitable where the bed consists of material which does not scour freely—such as clay or rock. The reason for this is that the stone is carried away as quickly as it reaches the toe, until the whole of the stone in the apron is exhausted. The bank face then becomes exposed just below water level and is quickly eroded by the current, wave action and river breathing.
>
> "An apron is also unsuitable where conditions are such that it does not launch satisfactorily because, (*i*) the angle of repose of the underlying material is steeper than that of the stone, or (*ii*) the material does not erode easily and evenly due to the bank consisting of alternate layers of sand and clay."

Inglis and Joglekar[5] have recorded the following opinion regarding guide banks:

> "Though this design (of guide banks) has been in favour for some fifty years (1945), it is far from perfect because stone has to be thrown in from time to time as apron stone gets washed away, and maintenance may be very expensive—as examplified by the case of the Hardinge Bridge, where nearly a million pounds was spent on repairs in a period of two years."

Figure 10 shows a guide bank with arrangement for its maintenance. Figure 11 shows a cross section of the guide bank on the Ganga

1. Lacey, G., "Stable Channels in Alluvium", in *Proceedins of the Institution of Civil Engineers*, vol. 229 (1929-30), paper No. 4736.
2. Inglis C. C. and Joglekar D. V., *op. cit.*, p. 348.
3. A stone that can be lifted by one man.
4. Inglis C. C. and Joglekar D. V., *op. cit.*, p. 372.
5. *Ibid.*, p. 380.

at Allahabad. It may be mentioned here that the work of artificial gorging does not usually stop with the construction of these banks. Training of the river further upstream by means of training walls (banks) or groynes is needed in most cases. And indeed, the breach of the right bank of the Hardinge Bridge on the Ganga in East Bengal in 1933, as mentioned before, though directly due to the failure of its apron, was later found to be indirectly caused by the stronghold constructed upstream at Sara, which so deflected the current as to make it severely attach the bank. Figure 12 shows the location of the banks and training works upstream of the Hardinge Bridge. The situation was much improved after the abandonment of the Sara protection bank.[1]

Denehy's groynes (India)

The construction of T headed groynes, known as "Denehy's groynes", for the purpose of river training and bank protection was first done in 1880 by P. Denehy at Okhla Headwork on the Yamuna river. The same type of construction was carried out at the Narora Headwork on the Gaga. Since 1887, the Narora groynes, as shown in figure 13, were built on both banks upstream, the left bank being an embankment to prevent spills from the river to the low land, and on the right bank downstream where the canal is located. They were spaced about half a mile apart for the 4 mi upstream from the railway bridge, and 16 mi downstream. The groynes proper were just an earthen embankment projecting from the main embankment, or the "marginal bund", as usually called in India, or the canal embankment, with the same free board above highest flood level. The head of the groyne took a T form, of which the front perpendicular arm, about 300 ft or more in length, was paralled to the current. A large portion of the T head was placed upstream of the groyne proper, so that it looked more like a reversed L than a T. The head sloped on both sides, with flatter slopes at the ends, and was heavily armoured stone. Reserve of "one-man" stone was also provided in the vicinity of the groyne.

The design of this kind of groyne aims at economizing the protection stone for the entire bank, as only 300 ft of it would suffice to protect half a mile length of the bank. The T-heads must be in a straight line or on a regular curve in the entire length of three or four miles, and should be spaced at half mile intervals. They must not be placed closer than this and no intermediate groynes should be introduced later on, for if the Denehy groyne is incapable of training a river or protecting a work at half-mile intervals, it loses the only advantage the inventor claims for it over the more efficient Bell's guide bank, that is cheapness.

1. Rao, T. S. N.: *History of the Hardinge Bridge up to 1941*, The Manager of Publications, Delhi, India.

Denehy's groynes were used both for river training and bank protection as demonstrated by the Narora example. They were used in a series as most types of groynes should not be used singly. Spring's proposal of building a stronghold at the upper-most end of the system, if nature could not provide one, was certainly a safe measure for long spurs of this type—although Denehy's experience at Okhla and Narora did not find them necessary. These groynes can be constructed efficiently on wide rivers only.

Bandalling (India)

Bandalling is an ingenious system of river training practiced since olden times on the rivers Ganga and Brahmaputra in India, designed to confine the low water flow in a single channel, with a view to maintaining a suitable depth for navigation.

A "bandal" consists of a frame-work with bamboos driven into the river bed and set 2 ft apart with horizontal ties and supported by struts placed at every 4 ft. To this bamboo frame-work, "jhamps", are tied with coir rope to the horizontal tie at water level. The bamboos used on the framework are usually 10 to 20 ft in length and the "jhamps" are made of bamboo mats from 2.5 to 3.3 ft (0.75-1.0m) wide strengthened at the edges by strips of split bamboo.

"Bandals" are placed at an angle of 30 to 40 degrees, inclined downstream to direct an additional volume of water into the desired channel and to increase the velocity in it. They do not stop the flow of water but merely check it—this causes the sand to be raised and carried forward along the channel, or carried outwards under the "jhamps" and deposited in ridges parallel to and behind the "bandals." Thus a channel confined between "bandals" is formed, with sand-banks on either side, and thus the whole discharge of the river is directed through this channel.

Nanal[2] plantations in India may also be considered a sort of river training. Nanal is a plant of cane type. It grows to a height of about 10-15 ft of which the bottom 4 or 5 ft is about ½″ to 1″ in diameter and is hollow like bamboo. The top portion has leaves like sugar cane, 3 to 4 ft long. Nanal grass planted along river banks will take root and sprout quickly. The growth of nanal checks velocity and induces deposition of sediment along the river banks.

Both bandalling and nanal plantations can be used to stop bank erosion.

2. A grass which grows to a height of 10-15 ft (3-4.5m), and bears a silvery grey, or white, feathery flower; something like pampas grass. Used for river conservancy works.

The improvement of the Whangpoo river and the Hai river (China)

The Whangpoo is the lowest of the tributaries joining the Yangtze and is the navigation highway to the port of Shanghai. The distance from its mouth to the harbour at Shanghai is about 33 km (20.6 mi). The mouth was obstructed by a bar (Wooseng Outer Bar) which, in 1905, was as shallow as 15 ft at the time of extraordinary low tides. Three miles upstream the river divided into two channels; one had a depth of only 8 ft (2.4 m) and the other (Wooseng Inner Bar) of 10 or 11 ft (3 or 3.3 m) at the lowest low water. Above that, the channel was in some places too wide with shoals rising at several points. In the harbour itself the deep channel was too narrow.

The work of improvement of the Whangpoo was begun in the year 1906 by the construction of a massive training wall at the left entrance, which deepened the outer bar. The inner bar was eliminated by the diversion of the river from one branch to a more direct course. The depth at the lowest low water was improved from 11 ft (3.3 m) to 15 ft (4.5 m) throughout.

The success of the first attempt led to the undertaking of over-all improvement in later years. The work was begun by laying normal lines 1,400 ft (420 m) apart at the upper end of the harbour, increasing the width to 2,400 ft (720 m) at the mouth. To guide the water between these lines, training walls had to be built in many places, of various types, with piles, brush-wood mattresses, caissons, stone etc., and also groynes at convex banks. At the division of channels by the island Gough, one of the branches, the so-called ship channel was closed by heavy dams, and the other branch was made into a first class waterway. In two places the river was too narrow and was widened to the normal width by dredging. Very large areas were reclaimed by the general narrowing of the whole channel, with the result that the deep channel became wider, the cross-sections became slightly larger in area, and the tidal current ran unimpeded from the Yangtze through the mouth of the Whangpoo in a gently curving course. In 1933, the entire channel was deeper than 24 ft (7.2 m) at lowest low water with a tidel range of 7 ft average, (max. 14 ft or 4.2 m). Ocean-going ships can now arrive at Shanghai without difficulty. Figures 14 to 16 show the river in the year 1906, 1911, and 1933, and also the location of most of the works. The two guide banks at the mouth were of massive breakwater type. Most of the training walls and groynes were constructed on fascine mattress foundation.

Another example of river training work in China is the Hai river near the Port of Tientsin in North China, which, when measured along the river, is 90.4 km (56.5 mi) to the sea. The river was extremely tortuous before improvement was begun in the year 1901, with many channels branching out from the main tidal channel. The result was that the tidal range at Tientsin was extremely small, barely a foot or so on the average. The navigation was also hampered by sharp bends and frequent turnings. Six cut-offs were executed during the years 1901-1923, resulting in a reduction of 25 km (15.6 mi) in the length of channel, and an increase of tidal range to about 6.48 ft (1.9 m) at Tientsin. Ships, which formerly could not be accommodated in the harbour, were able to come up the river to Tientsin. Three other cut-offs were awaiting execution, of which the longest—"Koku cut"—had already been started during the second world war.

The Hai river is the only exit for its five big tributaries which join it just above Tientsin and one of these, the Yungting river, is noted for the amount of silt it carries. It shows how river improvement may fail, if Fargue's laws are not followed. The "fifth cut" was made too straight by mistake. Whenever a large amount of silt came down the Yungting river, this reach got silted up very quickly, sometimes by 5 to 6 ft (1.5 to 1.8 m) in a day, forming a bottle-neck for navigation to the city of Tientsin. Revision of the curvature by means of permeable groynes of a very light type was tried but the progress was extremely slow. Figure 17 shows the cut-offs executed and proposed for the Hai river.

River training without embankments (Burma)

The few examples of river training practice mentioned above are all of large rivers. A special form of training works on small flashy streams, the so-called "river training without embankments", or rather building up of river banks by training works, in Burma, will now be described.

The background of this type of training works is a group of small streams flowing eastward from the Pegu ridges into the Rangoon river like herring bones, with drainage areas varying from 40 to 230 sq mi (103.6 to 596 km^2). Another group of rivers lying east of the ridge flows westward, and is composed of the tributaries of the Sittang river with catchment areas varying from 25 to 352 sq mi (64.8 to 912 km^2). The maximum discharge of these varies from 220 to 400 cu ft/sec per square mile (2.6 to 4.8 $m^3/sec/km^2$). The streams carry about one per cent of sediment by weight during flood, and this is essential to the success of the method adopted for training. The training is done for improving the streams for logging as well as for facilitating their passage through the bridges on the railway and road lines, and for reclamation of farm lands.

The process involved consists firstly of demarcating the alignment following a depression. Stake fences, consisting of bamboos 5 to 6 ft (1.5 to 1.8 m) long, spaced 10 in (25 cm) apart, are driven parallel to the selected centre lines at a pre-determined distance. Top of the fence is secured by horizontal bamboos lashed 6 in (15 cm) below the tip. Where stronger fences are required, for example where the line passes through a depression or the slope is steep, more rows of fences are provided. Sometimes as many as 5 rows are installed on sloping ground with each row driven closer together.

Banks are formed after the passage of floods over these fences, and a second fence is added at the top of the new bank. The process is repeated until a deep channel is formed with high natural banks on both sides. The bank formation should be carried out gradually from upstream downwards. Work in the first season is usually limited to 1½ to 2 mi (2.4 to 3.2 km), and extended by about 1 mi (1.6 km) each season. The result is very beneficial. Logging has been increased from 10,584 to 50,640 logs annually, while 50,000 acres (20,000 ha) of swamps have been converted into fertile lands with another 60,000 acres (24,000 ha) improved to yield 2.5 t of paddy per hectare per year instead of the 0.75 t before improvement, to say nothing of the uninterrupted communications and diminished flood damage. Same type of construction is used for closing breaches and building groynes. A general arrangement of the work is given in figure 18.

More examples of river training in the region

River training with permeable groynes has been tried with more or less success in this region. One instance was the failure of hurdle type groynes on the river Indus at Dera Ghazi Khan city, where Denehy's groynes failed to deflect the current. Three hurdles of the Mississippi type were then built in the year 1900-01, aggregating a total length of 5,100 ft (1,530 m) which were all knocked out during the flood season and only a short one 1,250 ft (375 m) long remained intact. However, permeable groynes have been successfully built on smaller rivers in India and China. Groynes described by Harrison,[1] of the lattice type, made of piles and cross timbers with their ends pointing upstream making an angle of 30 degrees with the line of flow, have been successfully used in India. Hurdle type groynes were tried on the Yungting river with success. These hurdles were intended to form a part of the whole project of river regulation described later.

River training above a bridge has the single aim of passing the flood flow smoothly through the bridge openings without causing damage upstream. (The narrowing of a river at a bridge site will no doubt alarm the inhabitants upstream for possible increase of flood heights. It should, therefore, be carried out carefully, so as to eliminate any opposition, even if only psychological fear may exist). Training of river above a headwork, however, is manifestly different as it requires the discharge of flood water over the weir as uniformly as possible and at the same time a channel supplying low water to the intake-regulator must be kept open at one or both banks. This double aim makes the problem much more difficult than the construction of guide banks. On the Narora headworks, in order to have floods well distributed over the weir, four perpendicular training walls were constructed up-stream of the weir which divide the latter into five compartments. The so-called pitched island, an armoured single pier curved to suit the flow with its tip pointing up-stream, had been experimented in models for the purpose of drawing the main river channel on to itself and holding the river there. Actual construction of a pitched island was also tried at 4,500 ft (1,350 m) upstream of the Suleimanki Weir on the Sutlej river in the Punjab.[2] This is very much like the "Li-tui" or the separate ridge above the Tu-kiang headworks on the Mien river in Szechuan, China, which trains and divides the river into two separate channels to supply two intakes from the river. But the latter had served so successfully that only occasional repairs were needed since its first construction about the third century, while the pitched island on the Sutlej did not answer the intended purpose. Perhaps further study and trial may reveal its usefulness.

In view of contracted rivers becoming deeper in course of time, river engineers in China had schemes for the regulation of long stretches of the Yungting and the Yellow rivers, both of which tend to raise their beds perpetually, menacing the existence of their flood embankments. The two schemes are very much alike, as the two rivers are very similar. River contraction by training walls, groynes and closing dikes are proposed together with lighter type of planted willow growth. Only a small and scattered portion of the scheme, comprising permeable groynes, had been carried out on the Yungting river as mentioned above. Although the result remains to be seen, it is certainly worth a trial, especially on a smaller river like the Yungting, so as to find another way of flood abatement by increase of channel capacity.

1. Harrison, J.L.: "River Training Works", in *The Royal Engineering Journal*, vol. LV, March 1941, London, p. 51.

2. Montagu, M.R., and Uppal, H.L.: "River Training and Control by Pitched Islands" *Proceedings of the Punjab Engineering Congress*, 1945, Vol. XXXIII, paper No. 275.

Cost of River Training

Works of river training vary so greatly as to purpose and location that any comparison between the cost of one with another, except for navigable waterways of like capacity, is not only impracticable but also unnecessary. Different methods of river training suit different rivers, and the benefits derived from training may be different. For instance, land reclamation from the improvement of the Whangpoo river was a return of great value while any land reclaimed at irrigation headworks from the training of rivers would be just a trifle in value as compared with the cost of river training.

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Chapter III

BANK PROTECTION

PURPOSE AND SCOPE OF BANK PROTECTION

In a broad sense, a bank can be defined as the sloping surface of land coming into contact with a body of still or flowing water. Bank protection, therefore, includes any protective work that aims at maintaining the stability of land against the action of water. Under such a broad definition, bank protection includes works of shoreline protection along the sea and lakes against waves and littorial drift; protective works along navigation canals against waves created by passing vessels, and protective works along embankments and banks along rivers for the purpose of flood control, navigation, or river training.

The present study limits its scope of protective works only along rivers. Because of the variation of the force of water acting at different parts of a cross-section of a river and because of the variation in the duration for which the various parts in a cross section are in contact with water, one can divide a river bank into three parts. viz: (1) Embankment, i.e. the sloping surface of embankments facing the river; (2) Upper bank, or the portion of the river bank that is located above the low water level and below the foreshore level; and (3) Lower bank, i.e. the portion of bank below low water level extending quite a distance horizontally to the river bed which is usually called the toe.

Embankment

Embankments are usually set back away from the main channel of the river and are therefore usually not subject to very strong currents. As embankments are constructed on the flood plain or foreshore of a river, they come into contact with water only during floods. Problems encountered in the protection of embankments are the protection of the surface against currents of relatively slow velocity and the maintenance of stability as a result of infiltration of water into the embankment.

Upper banks and lower banks

The upper bank together with the lower bank forms the side of the main channel of rivers and streams. The upper bank is subject to relatively strong current. The lower bank, being the foundation supporting the upper bank and always under water, is indeed the portion most susceptible to erosion and requires the most of attention. Recession of banks is usually due to erosion of the lower bank, in particular at the toe. Bank recession approaching the embankments often causes breaches of the latter.

When embankments are constructed very near to the main channels of rivers they become a part of the upper bank. In torrential rivers where embankments are constructed to confine the flood flow and where the foreshore is subject to strong erosion during floods, special protection is required to be provided at the toe of the embankment as well as on its sloping surface. In such cases, an embankment acts more or less like the lower bank of an ordinary alluvial river. Such rivers are to be found in the coastal plains of Japan and Taiwan and to some extent in the North China plain, like the Yellow River.

BANK RECESSION AND THEORY OF EROSION AND SCOUR

Causes of bank recession

A bank may fail owing to any one or a combination of the following causes:

(*a*) Washing away of soil particles of the bank by current or waves; this is called erosion;

(*b*) Sliding due to the increase of the slope of the bank as a result of erosion and scour;

(*c*) Undermining of the toe of the lower bank by currents, waves, swirls, or eddies followed by collapse of overhanging material deprived of support; this is called a scour;

(*d*) Sloughing or sliding of slope when saturated with water; this is usually the case during floods of long duration;

(*e*) Sliding due to seepage of water flowing back into the river after the receding of flood; the internal shearing strength is considerably decreased owing to saturation and the stability is further decreased by the pressure of seepage flow;

(*f*) Piping in a sub-layer due to movement of ground water to the river which carries away sufficient material with it.

The way banks fail under (*a*), (*b*) and (*c*) may be attributed to direct erosion or scour. Failures under (*d*) and (*e*) may be said to be due to reduction of internal strength, and those under (*f*) due to foundation failure. The last three are a result of saturation and seepage of water.

Generally speaking, the stability of a sloping bank under the action of water is subject firstly to the static head of water which induces seepage and saturation and secondly to the erosive force of running water which changes the form of the slope and in turn affects its stability. A brief description of the theory of erosion and scour is given below and this is followed by a discussion of seepage and saturation.

Theory of erosion and scour

Erosion of river banks and scouring of their bed take place when particles composing these parts are acted upon by forces sufficient to cause them to move. It has been shown in sub-section *The tractive force theory* that for a straight channel of uniform depth the acting force of running water on the river bed, or the average tractive force, is equal to γRS, where γ is the unit weight of water, R the hydraulic radius and S the slope. The tractive force which is just sufficient to set the bed material of a certain kind into motion is called the critical tractive force. If the river bed material is non-cohesive, there is a definite relation between the critical tractive force and the mean diameter and specific gravity of the river bed material. Reference may be made to table 1 and figure 2 wherein the critical tractive force for different sizes of bed material is given.

For a meandering river, the actual tractive force will be larger than the mean value given by the expression γRS on account of the shifting of maximum velocity towards the concave bank as well as the development of a secondary current. The percentage of increase of tractive force, or the decrease of river bed resistance as compared with a straight channel, suggested by Lane for different degrees of sinuosity of channel, is given in table 2, (p. 15).

The statement made above refers to the erosion of the non-cohesive bed of rivers. On a slope, as pointed out by Fan,[1] the resistance of a sediment particle to motion is reduced by the sliding force of the particle itself on an inclined plane due to its own weight. If K is the ratio of tractive force required to move a particle on the inclined plane to that required to move the same particle on a level bottom, Fan shows that for non-cohesive material K is affected by the angle of repose θ of the material and the angle of the side slope ϕ and can be expressed by the following equation:

$$K = \sqrt{1 - \frac{\sin^2\phi}{\sin^2\theta}} \qquad (25)$$

A set of K curves for various values of ϕ and θ is shown in figure 19, which is valid for non-cohesive material only.

Another important point to be considered is the distribution of tractive force on the side slope of a river bank or canal, which has been worked out by Olsen and Florey, using the method of membrane analogy.[2] Experiments conducted by Olsen and Florey covered trapezoidal cross-sections of various side slopes and of various ratios of bottom width W to depth of flow D ranging from 2 to 8, the latter being considered applicable to channels of great width. The tractive force is zero at the bottom of the side slope, increases to a maximum at a place somewhere between 0.2D to 0.3D above the bottom, and decreases to zero at water surface. The values of the maximum tractive force acting on the side slope, when expressed in terms of γDS, are given in table 4:

Table 4

Maximum tractive force on side slope of trapezoidal channels expressed in terms of γDS

Side slope	Ratio of bottom width W to depth D		
	2	4	8
1:2	0.760	0.770	0.770
1:3/2	0.735	0.750	0.760

1. Fan, C.H.: "A study of cross-sections of equilibrium canal", in *Chinese Institute of Hydraulic Engineering, Monthly*, No. 1, Vol. 15, October 1947 (Nanking, China), p. 71.

2. Olsen O.J. and Florey Q.L.: "Sedimentation studies in open Channels etc." US Bureau of Reclamation, *Structural laboratory report* No. SP-34, August 1952. Denver.

An example is now given to illustrate the use of the formula (25) and diagrams (figure 19). An indefinitely wide channel has a mean depth of 10 m (33 ft) and a slope of 0.0002. The prevailing tractive force of running water is then equal to 2 kg/m² (0.412 lb/sq ft). If the channel is very sinuous, then from table 2, the actual tractive force would be $2.00/0.60 = 3.33$ kg/m².

If the bank has a side slope of 2 (horizontal) to 1 (vertical) and the material has an angle of repose of 35 deagrees then from figure 19, $K = 0.63$ and from table 4 the maximum tractive force on the side slope is 0.760 of γDS. The actual tractive force would therefore be $3.33 \times 0.760/0.63 = 4.00$ kg/m². From figure 2, it is found that particles of about 55 mm diameter, that is a mattress of cobbles, will be required to resist erosion.

The above derivation is for non-cohesive material only. If the material is cohesive, size of particles may be reduced. For irregular and very turbulent flow, such as that below a dam or weir, or wave wash, the actual force may be much greater than the tractive force thus computed. So is the case of upper or lower bank subject to wave wash or any eddy current erosion where forces acting on the bank material are very different and cannot be figured out by the used of tractive force theory.

Localized scour

Systematic experiments upon the rate of scour of a sediment bed were carried out by Rouse.[1] These experiments were conducted with a vertical sheet of clear water falling upon a sediment bed. The resultant findings were (see figure 20):

(a) The depth of scour D_s in a uniform material is dependent solely upon the thickness T of the sheet of falling water, velocity of the jet V, the fall velocity of the sediment particles w, and the duration of the scouring action (figure 20).

(*b*) The relative rate of scour produced by a given jet D_s/T at a given stage (time) depends only upon the ratio of jet velocity to fall velocity V/w, approaching zero as this ratio approaches unity in accordance with the sequence of lines shown in figure 20.

(*c*) Selective sorting of grade material occurs, so that with a wide variation in size of bed material the bottom of the hole gradually becomes paved with a progressively coarser material, thus decreasing the effective fall velocity of the sediment and reducing the scour rate as indicated by the broken line in figure 20.

(*d*) If the sediment is supplied to the jet at exactly the same rate at which a clear jet would scour, the scour hole will be stabilized; by inference, an increase in the rate of sediment inflow would cause deposition and hence gradual filling of the hole until equilibrium is again established.

These findings are directly applicable both to experimental models of prototype structures and to the design and maintenance of the structures themselves. From the exponential nature of the scour function, a limiting depth of scour cannot be expected either in the model or in the prototype, unless selective sorting itself eventually paves the hole with material of a non-scourable size, or unless by progressive enlargement of the scour hole the local velocities are eventually reduced below the fall velocity of the bed material. Also, the use of model sediments to determine rates of depth of scour can only give qualitative results unless similarity in the rate of flow velocity V to fall velocity w is also attained.

The fall velocity of sediment particles in water under their own weight with specific gravity = 2.65 is given in figure 21 for different temperatures of water.

An empirical formula for scour along banks given by Inglis and Joglekar has been given in equations (20) to (22). In addition to this, Inglis and Joglekar[2] give the following formulae:

Scour at straight groynes, facing upstream, with steep sloping head ($1\frac{1}{2}$ to 1)

$$D_s = 1.8 \left(\frac{Q}{f}\right)^{\frac{1}{3}} \qquad (26)$$

and with long sloping heads (1 in 20)

$$D_s = 1.3 \left(\frac{Q}{f}\right)^{\frac{1}{3}} \qquad (27)$$

for other groynes along river bands

$$D_s = (0.81 \text{ to } 1.8) \left(\frac{Q}{f}\right)^{\frac{1}{3}} \qquad (28)$$

where D_s is the depth of scour in ft, Q is the discharge in cu ft/sec and f is the Lacey's silt factor.

Seepage and saturation

When water level rises with the increase of flood stage, banks and embankments originally above water level become drowned and, depending upon the duration of flood, they will be partially or fully saturated with water. In case of silt

1. Rouse, Hunter: "Criteria for Similarity in the Transportation of Sediment", *Proceeding Hydraulic Conference* (University of Iowa Studies in Engineering), 1940, p. 33-39.

2. Inglis, C.C. and Joglekar D.V., op. cit., p. 348.

and silty sand, the angle of shearing resistance may be reduced to 70 or even 50 per cent of its original value by saturation. If the angle of the sloping surface is steeper than the reduced angle of shearing resistance, slough is the sure result of such saturation. This is much more prominent during the receding of flood water, as the seepage flow reverses its direction of flow. The seepage water may carry with it certain quantity of fine particles which reduces the shearing resistance of the bank material. If the seepage continues for a long time, the bank may fail by passing into semi-liquid state or by losing certain thin layer of supporting materials.

Seepage is also dangerous during high flood stage if the bank is in the form of an embankment and the slope facing the land side is too steep to cover up the seepage line. The washing away by seepage water of the finer particles may also result in a failure of the sloping surface by shear.

Piping is a case of seepage flow where the weight of the materials at the exist is not sufficient to withstand the uplift pressure exerted by the seepage flow, or where seepage finds its way through a weak spot or a crack.

METHODS OF BANK PROTECTION

Classification of bank protection works

Bank protection works may be broadly classified into two groups: direct protection and indirect protection.

Direct protection includes works done directly on the bank itself such as: slope protection of embankment and upper bank and toe protection of lower bank against erosion, and grading of sloping surface or provision of drainage layers to ensure stability against seepage and saturation. As such works cover continuously a certain length of the banks, they are also called continuous protection.

Indirect protection includes such kind of works as are not constructed directly on the banks but in front of them with a view to reducing the erosive force of the current either by deflecting the current away from the banks or by inducing deposition in front of them. This is usually done by means of groynes spaced at a certain distance apart and is therefore also called non-continuous protection.

Under both direct and indirect protection, some differentiation can be made according to the durability of the works among: (1) temporary protection to check erosion at times of emergency, (2) semi-permanent protection using brush wood, bamboo, sorghum or wire mesh that will last a comparatively short time, and (3) permanent protection including the use of more permanent materials like stone, concrete and also timber or brushwood mattresses if kept always under water.

Summing up, the various kinds of protective works can be classified as follows:

(*a*) Direct protection
- (*i*) Slope protection of embankments and upper bank
- (*ii*) Toe protection of lower bank

(*b*) Indirect protection
- (*i*) Deflection of current
  - Bank heads
  - Repelling groynes
- (*ii*) Deposition of sediment
  - Sedimentation groynes.

Direct protection

(*a*) *Slope protection of embankments and upper banks*

River side slope of embankments and upper banks of rivers, if not subject to strong current, can be well protected from erosion by a vegetal cover, either by turfing or by sodding or by other natural growth. Low growth of shrubs and willows is by far the most effective cover, but they require a longer period to grow. In such cases, a temporary cover with mattresses of woven willow brushes is often necessary for the initial period of one to two years. After the natural growth has taken place, it just needs cutting once every year to keep the brush-wood from growing into tall trees. If extensively planted along the banks, the return from brushwood, especially where mattress work is largely in use, might be considerable.

If the current near the upper bank and embankment is quite strong particularly during floods, paving of slopes with materials that can resist erosion is necessary. Many types are cited in the next section as well as in the next chapter. Temporary covering by brushwood or lumber mattresses (usually the woven type) and weighted down by stones is used for emergency purposes or for brush growth as mentioned. Paving with broken stone is the most common type used everywhere. All pavings must have a firm connection with or be made continuous with the toe protection. This will be treated separately under toe protection in the next section. In case the bank is exposed to wave action, like the face of a sea wall or the bank slope of a ship canal, the paving will have to resist not only the onrushing of wave wash, but also the sucking caused by the receding waves. Specially tight construction is therefore required to prevent the earth under the paving from being sucked out through the joints.

All sodded, vegetal covered, or broken stone pavement must be graded before-hand to a slope at least equal to or flatter than the angle of shearing resistance of the soil under water. Drainage behind the paving is sometimes necessary in order to prevent the return seepage flow carrying away fine particles of soil.

As an indication of the relative strength of different kinds of revetment, values given by Shoklitsch[1] and Sih[2] are reproduced in table 5. The values of critical tractive force given apparently refer to the tractive force of the river as calculated by the formula γRS and to the slopes having the conventional inclinations.

Table 5

Type of construction	Critical tractive force (kg/m^2)
Coarse sand between wattles	1.0
Gravel between wattles	1.5
Grass sodding	2.0—3.0
Wattles, parallel or oblique to currents	5.0
Mattresses	7.0
Brick paving	10.0
Stone paving	16.0
Random-placed, riprap of large stones	24.0
Dry masonry on timber grillage	60.0
Concrete pavement	60.0
Reinforced concrete pavement	80.0—100.0
Crib with stone	up to 150.0

All protection work covering earthen banks with a slope steeper than the angle of shearing resistance of earth may be classified under the type "retaining wall." Thus the protection work would have to be strong enough to resist pressure from the earth behind it and at the same time to resist the erosive force of water in front. Examples of both temporary and permanent structures of this type are abundant.

Temporary retaining walls may be built of closely driven piles, piles and timber boards or sheeting, piles and brushwood packing, and sorghum stalk packing; the last named will be described more fully in the section on the Yellow River (p. 42-43). All these may last for one to a few years and may be replaced by more permanent structures later on, although sorgham stalk packing is usually replaced by new packing of the same material. Permeable pile fencing or crib work is sometimes built along the bank for the purpose of catching some silt to stop erosion.

Masonry retaining walls built on pile or other forms of foundation are heavy and expensive but are effective in protecting the banks against wave wash. They are used for sea walls, or flood walls at the water front of a city where land is particularly valuable. For sea walls, different profiles may be used with the object of guiding the waves so as not to splash over the top of the embankment. At port wharves and landings on a river, the vertical type of retaining wall is more favourable than the sloping one.

Besides being heavy, masonry retaining walls for banks protection have another drawback in that they are not feasible for deep foundations below low water level. The daily construction time is shortened to a few hours only, particularly on tidal rivers. The development of steel sheet piling enables engineers to put the foundations in the bed as deep as desired and to carry out the construction for as long as twenty-four hours a day. The finished work is neat and strong, and as durable as a masonry wall. Reinforced concrete sheet piles are also in use, but they are usually not as water-tight as steel piles, except the "Franki" piles, and also they are not so durable in certain kinds of sea waters. Special sheet piles, made of steel mixed with copper, have been found to be most satisfactory on the Suez Canal.

(*b*) *Toe protection of lower bank*

Protection of lower bank differs from that of the upper bank in this respect that, in addition to the fact that lower bank is constantly under water, its toe is liable to scour. Protection of lower bank starts somewhere near the low water level and extends toward the river up to a distance corresponding to the possible depth of scour. This scouring, according to Spring, can be classified into (*i*) normal scour, (*ii*) abnormal scour, (*iii*) extraordinary scour and (*iv*) suction scour. Normal scour in an ordinary straight channel is caused by the high velocity of a river during floods. This is controlled by the critical tractive force as elucidated before. The abnormal scour is usually found at the concave banks where

1. Schoklitsch, A: *Hydraulic Structures* (English translation), vol. II, p. 1061.
2. Sih, Lee-Tang, "Embankments and Bank Protection of the Taiwan Rivers", in *Proceedings of the Regional Technical Conference on Flood Control in Asia and the Far East* (*Flood Control Series No. 3*, UN publication, sales No. 1953. II.F.1), p. 101.

the current, during flood times, strikes the bank obliquely with a high velocity and digs deep along the bank just like a water jet. The extraordinary scour is caused by swirls or whirlpools. Bed material is picked up and carried forward by water where bottom velocity is adequate to move the individual particles of the more or less incoherent bottom. This has been described in sub-section "Localized scour". A fourth type of scour at the toe may be called suction scour, which is caused by the impact and suction of the translation maves created by movement of boats or by tidal action. Therefore, toe protection must be extended to possible depth of scour, even if such scour may not be realized for years to come and the protection must be as flexible as possible so that it will cover up the bed when any scour does take place.

To meet the cases cited above, four different ways are now used in practice. The most common method is to provide a flexible apron extending to the river bed as far as needed. This may include all kinds of mattresses, as well as the falling apron used in India. The second method is to provide enough material, usually stone, as a reserve, which may be dumped into the hole to stop further scouring. Where the current is specially strong, stones packed in different kinds of cages such as brushwood or bamboo sausages, wire mesh sausages or mats, etc., may be used. This second method may be cheaper in first cost but is less reliable in time of emergency, especially when extraordinary scour occurs during times of great floods and storms, as no one can predict how far and how deep the scour would extend. It is too hazardous to take the chance of saving a little in first cost, with the probability of exhausting the material provided and the entire loss of an important structure.

The third method of toe protection is by the provision of an impregnable curtain of retaining walls or sheet piles, etc. This is quite expensive, especially where deep scouring is expected. Steel sheet piles can answer the purpose best. Reinforced concrete sheet piles (except the "Franki" concrete piles) or wooden sheet piles are usually not water-tight and the flow of water through sheet piles may weaken the foundation for the superstructure. Closely driven round piles can perform the work where scouring is moderate. Retaining walls built at the foot of a slope serve both as foundation for the pitching above and as toe protection below; but they cannot penetrate deep below water, and another form of toe protection in front of them may be necessary.

The checking or elimination of scouring forces is the principle adopted for the fourth method. This may include the construction of submerged sills or spurs below deep holes in front of a concave bank. This not only corrects the depth of the river but also stops further deepening of the bed and therefore protects the toe of the bank. In different countries, many types of current slackening devices, including the growth of weeds, have been developed, some of which will be described in detail in subsequent discussion of river works of various countries.

Indirect protection

Indirect or non-continuous protection of banks is in general cheaper than direct or continuous protection, but is not always so effective. Temporary protection like overhanging trees, sinking trees, floating booms or brushwood fascines hanging on pile structures are mostly for spot protection when scouring is endangering the river embankment or other structure. For permanent and semi-permanent protection, three types of non-continuous works are used in practice; (*a*) bank heads, (*b*) repelling groynes, and (*c*) sedimenting groynes.

(*a*) *Bank heads*

Bank heads are really artificial bluffs that deflect current away from the bank. Eddies and swirls are created at the downstream side of the bank head and if eddies extend further downstream, the bank may be caved much more severely than without the head. The bank head then becomes an attracting instead of a repelling structure.

Bank heads may be built permanently or semi-permanently. Permanent bank heads are built of stone or stone laid on a mattress foundation, spreading fan-like from a centre point on the bank with a radius depending upon the size of the river. Radii of 30 m to 100 m are not uncommon. A flat slope of not less than 1 in 3 to 5 is usually provided, with a strong protection at the toe. To ensure the protection of a certain length, the river bank up and down stream of the head must also be protected. Semi-permanent type of construction by means of sorghum stalks and earth, just like bank retaining works described later, have been practised on the Yellow River. They are called grind-stone dikes (Mo-puan Sao) for larger ones, or fan dikes (Shain-mien Sao) for small ones.

Bank heads, when properly located and strongly constructed, are sometimes very effective in deflecting the current away from the river bank and protecting it from erosion. But this is not always the case.

If they are required to be closely spaced, it will make the construction much more costly than continuous revetment. Also, because of the sudden deflection of current, the protection of one side of a river may be at the cost of the other side and may greatly complicate the problem.

Since 1897-98, eleven bank heads were constructed on the Missouri river, each of a 100 degree arc, with a radius of 350 ft. But these structures were very vulnerable with the swirling water attacking the exposed banks above or below them, and also the unprotected river margin beneath them. River bed in front of one bank head was scoured to a depth of 35 ft (10.5 m) below low water, which increased to 64 ft (19.2 m) a short distance away. Caving was continuing between the bank heads, but to a lesser extent than before, in one case, the recession being 500 ft (150 m) in two years.[1]

(*b*) *Repelling groynes*

Repelling groynes have the main purpose of deflecting currents away from the bank or of shielding the bank with a strong arm in front of it, although sediment may be caught above or below the groynes. These groynes must have a strong head to resist the direct attach of a swirling current and are usually built of stone or stone-pitched earth embankment, with paving or other forms of protection at the toe of the slope. The scour gradually diminishes from the head to the bank and the protection of the slope and the apron can be reduced accordingly. As the heads are always under attack, much repair may be required from year to year.

Repelling groynes may be of permanent, semi-permanent or temporary construction.

Groynes may be inclined upstream or downstream, or may be normal to the bank. Recent practice tends to favour the upstream inclination, making an angle of 15 to 30 degrees with the line normal to the flow. However, on the Missouri, a downstream deflection of about 15 degrees is favoured principally to offset the damaging attacks of drift and ice.

Repelling groynes are usually constructed in a group, spaced like groynes used for river training. The first or upper most groyne should either be just a bank head attached to the bank, or it must be very strongly constructed, as the attack on this groyne is usually the severest. A single repelling groyne to protect a bank is seldom built, but examples are many especially when one considers that widely spaced groynes act singly.

(*c*) *Sedimenting groynes*

"Sedimenting groynes" is a name coined just to distinguish these from the repelling type. Most of the permeable types of construction work fall in this category and one could easily find that permeable groynes are not very well adapted for clear-water rivers. Permeable structures are generally of semi-permanent construction, although reinforced concrete has been used in some places. As sediment is accumulated between the groynes, the foreshore is more or less permanent, so that there is no need of using too durable a material. Sometimes, especially for toe-protection, these groynes may be constructed very low, and after the accumulation of silt to that level they can be raised to a higher level. This method of gradual raising is also used in coastal protection, where short groynes, spaced close together, are used for the purpose of collecting littoral drift in order to establish a permanent beach in front of a sea wall, etc. These groynes may be of the solid or permeable type depending upon the condition of the current and the waves. If of solid construction, groynes must be made as smooth as possible with the current so as not to induce violence of action. The gradual increase of height not only provides smoother flow of current passing them, but also saves a great deal of first cost.

The requirements of indirect protection are effective location and a form of structure that would answer the purpose of deflecting or slackening the current or silting up of the foreshore, with the material available without unduly high cost. If works of great extent are projected, model testing as well as construction of experimental structures is considered desirable to begin with.

Before discussing the various causes of failure of bank protection works it may be pointed out that any river works including river training, no matter how well they are designed and built, cannot fulfil their function without adequate maintenance. Many river works have been well designed and constructed, but have been neglected and even forgotten and blames of their failure have been incorrectly placed. It has been rightly pointed out by Snakenberg[2] that all types of engineering to water and soil require continuity of effort and that without adequate, continuous, wise and careful maintenance being first provided for, works for the conservation or control of water should not be proceeded with.

1. Van Ornum, *Regulation of Rivers*, *McGraw* Hill Book Co., 1916.

2. Snakenberg, Discussion on the paper by Grant, A.P. "Channel Improvement in Alluvial Streams", *Proceedings New Zealand Institute of Engineers*, 1948.

Failure of bank protection

Some cases of failure of bank protection works can be attributed to the failure of the toe, which is usually caused by (*a*) insufficient depth of protection; (*b*) insufficient weight of the protecting material and therefore its being carried away by the current; and (*c*) non-flexibility of the mattress. This failure may be experienced at heads of groynes or bank heads, or along a bank revetment work including retaining walls.

Bank head and groynes may fail by out-flanking due to (*a*) insufficient protection at the wing or (*b*) embayment by the river further upstream. Long groynes may be breached at their middle portion owing to infringement by a sharp-turning current.

Paved slopes themselves may fail because of one or more of the following causes:

(*a*) Insufficient thickness damaged by concentrated impact due to floating debris or trees
(*b*) Inadequate drainage at back of the bank
(*c*) Insufficient depth of paving
(*d*) Permeability of mattress or paving
(*e*) Insufficient layer of spalls or crushed stone under paving
(*f*) Insufficient bond between upper bank paving and protection
(*g*) Dislodging of mattress or paving blocks
(*h*) Piping out of lines from underlying soil due to inadequate filter blanket.

BANK PROTECTION PRACTICE IN THE REGION

Protection of Yellow River embankments

For thousands of years, the Yellow river has been subject to the constant danger of bank caving, which became particularly serious after the construction of its embankment system. Protection of embankments was therefore an old task for engineers on this river. Stone, bamboos, weeds, etc. had been used according to the earliest records of breach closure. As the river is in most parts far away from stone quarries, and the cost of transportation of stone by land (since the river is not a good navigable one) is prohibitive, a particular method of utilizing the local materials, the sorghum stalks, for bank protection and other river works has been evolved and perfected in the course of years. It affords a comparatively cheap and effective, but temporary, protection for river structures subject to erosion. In recent years, it has been gradually replaced by stone or brick work as easy decay of sorghum stalks became a constant source of drain on the limited funds available, but for emergency works, when other materials are beyond timely access, it is still used as a last resort.

The principal materials used in the above-mentioned method which is unique on the Yellow River are sorghum stalks and earth with ropes and timber piles as auxiliaries. Sorghum, a grain-bearing weed, is the main crop on the flood plain along the Yellow River. Its stalks are hardy sticks about 1 to 2 cm in diameter and from 2.5 m to 4 m in length, somewhat like bamboos though not so durable, and are used as building material for roofing, fencing, etc. of a temporary nature. They are also the main source of fuel and cattle feed in this region. For river works, they are therefore about as inexhaustible as the earth itself.

The method of construction of sorghum stalk works is as follows: A row of stakes 1½ m to 2 m long is first driven on the top of the embankment and on each stake a rope, made of weeds or jute, is fastened. The other end of the rope, which is about 30 to 100 m (100 to 333 ft) long, is fastened to a beam elevated at the middle of a barge over its whole length. The barge is called a fascine launch barge, of which the length is usually a little longer than a single piece of stalk work. When the work can be done on a dry river bed, a sort of shelf is used instead of a barge. Ropes form the bottom net of the entire work, on which rolls or fascines of sorghum stalks are put parallel to the direction of flow or the bank line. These fascines are bound by the same kind of ropes, usually at 1.5 m (5 ft) intervals and individual fascines are also fastened together with adjacent ones. When the layer of parallel fascines reaches the required dimensions in width and length and has a depth of about 2 ft, more stalks are added on to form a level surface, over which a thin layer of earth, about 15 cm (6 in) thick, is spread at the top. More fascines are then put above the bottom layer, and normal to the flow with their butt end facing the water. Stakes are driven to fasten the two layers of fascines. After enough weight of fascines and earth has been put on so that it may reach the level of water, the bottom ropes are loosened from the boat, usually begining from the downstream end; with enough workmen to hold the ropes and loosening them simultaneously but gradually, the mattress is launched on to the river bed. Thousands of workmen are required to tamp the launched fascine mattress to become solid, and piles are driven obliquely to the river bed, with ropes fastened back to them. A heavy layer of earth is usually put on to weigh it down. This is called a "pei" or a mat. Another row of stakes with ropes is then added and this process is repeated until another "pei" is put on top of the first. Several such mats can be added on until the required height is reached and a layer of earth is put on the top of it. The whole piece of work done in this manenr is called a "sao", although different materials like stone, brick, lime concrete and brush wood are used for construction of "sao", this was by far the most common method used in old days. Figure 22 shows the construction of a sorghum stalk "sao".

Many kinds of river works have been constructed by means of "saos" including groynes, bankheads, closing dikes, wing walls as well as bank revetment works. When used as bank revetment, one "sao" is overlapped by another at the upstream end like fish scales and the whole work is called a "fish-scale sao" (figure 23). These fish-scale saos are very effective in resisting bank erosion, but are very short-lived owing to the quick decay of sorghum stalks, which crumble to dust in a period of a year or two. Renewed construction must be done where the erosion continues to be severe. This is, of course, an expensive procedure in the long run. But as the river changes its course very frequently, temporary construction like sorghum stalk "sao" has the merit of being cheap and effective in a great portion of the work on the Yellow River, especially at the time of closing a breach. With the development of modern transport methods, stone can now be obtained much cheaper than before, and sausages or cylinders made of willow bushes and stone have largely replaced the sorghum stalk "sao" in recent years. Brushwood fascines had been used even to build permeable groynes with the aim of accumulating silt in the bays formed by these groynes. The plan of "Regulation of the Lower Yellow River" prepared by the Yellow River Commission, however, proposed to use brick paving and submerged crib work filled and overlaid with broken bricks for bank revetment above and below low water level respectively, with wooden piles driven along the foot of the bank to prevent them from being carried away by the current. Falling apron is also proposed for further development of the bank line. Sediment arresting dikes or silt dams, and fence type permeable works are also proposed.

More examples of bank protection in China

The embankments along the Chientang river and the tidal flats of the Hangchow Bay have been protected by various kinds of stone armour, but the main type has been the retaining walls resting on pile foundations. The wall is built mostly of stone blocks about 0.32 m $\times$ 0.32 m $\times$ 1.44 m (1.1 ft $\times$ 1.1 ft $\times$ 4.8 ft) with the length laid normal to the bank line and at a slope of about $1 : \frac{1}{4}$ on the river side. The height varies somewhat from 8 m to 10 m. These walls can stand the attack of waves as well as of bores which are created by tides due to the sudden elevation of the river bed at its mouth. The failure of these walls is primarily due to the scour at their toe, and much attention has been given to the protection of that part of the walls. Formerly, brushwood with piles had been used to protect the toe, but its life was usually short. Stone blocks laid on the bed with closely-driven piles about 5 m long had proved to be sufficient for wave action alone but along banks where bores were experienced, stone blocks had to be laid parallel to the flow with one block overlapping half the length of the other, and at some places, stone blocks, 1.44 m in length, had to be laid vertically against two to three rows of piles. Even this method of toe protection required frequent repairs, as the piles were attacked by marine borers and new piles were needed to replace those already eaten up, except where the blocks were laid vertically. The cost of laying stone blocks vertically was rather too much compared with the scanty funds available.

A special type of failure by deep scouring was observed by S. T. Hsu at the sea walls along the Chientang river in the Hangchow Bay. The failure of the apron along the wall led to a deep scour below the toe of the wall, which finally made its way under the base of the wall. The wall was supported on piles which, when exposed by scouring, were attacked by toredos or marine borers. The wall collapsed after the piles had been deprived of their strength through loss of the heart wood.

Groynes were constructed along the right bank in 1928-29 to protect the lands outside the embankment where a total area of about 3,000 to 4,000 ha (7,500 to 10,000 acres) had been lost in course of years. Three groynes of the mound type with rubble stones stopped the erosion and recovered a portion of the land. The longest groyne was more than 900 m (3,000 ft) in length. The restoration of land on the right bank, however, induced erosion on the left, which finally reached the walled embankment and destroyed quite a length of the old wall. Groynes were tried but with little success.

Groynes were used on the Yungting river to protect a newly closed breach in 1924 and were successfully maintained except the 800 ft (240 m) long one which was breached several times in the middle. Later, hurdles were built on that river with remarkable results.[1] Some types of solid groynes, consisting of an earthen embankment slightly inclined downstream to the flow and a hood-shaped head paved and protected with riprap, were used on the Che-sui river in Taiwan with some success.

Flood embankments in Taiwan, however, were largely built of cobbles picked up from the river bed and the facing was strengthened by embedding the cobbles in cement mortar. Except where the swift current brought down drift wood etc., and gave hard blows to the cobbles and eventually destroyed the embankment, there was little danger of erosion of the slopes of the embankment covered with cobbles. The most vulnerable part of the structure was therefore the toe. Various methods of toe protection were tried, including (*a*) wire cylinders or sausages, (*b*) reinforced concrete revetment with monolithic concrete apron, (*c*) concrete blocks, (*d*) fascine mattresses etc.; but by far the most effective and cheap type was the use of wire cylinders.

Cylinders made of woven bamboo and filled with stone had been used but the life of these cylinders was only three to four years and they are now used exclusively for temporary protection. Wire cylinders of elliptical cross-section varying from 0.67 m to 1.00 m (2.2 ft to 3.3 ft) in width, 0.40 m to 0.60 m (1.3 ft to 2.0 ft) in height and 10 m to 15 m (33 ft to 50 ft) or more in length are made from a woven net of No. 8 galvanized wire. The cylinders are then transported to site, filled with cobbles and placed against the slope and the toe, with their lengths normal to the direction of flow. Any scour would incline the cylinder and protect the bed and thus stop further scouring. Figures 24 and 25 show the wire mesh cylinder as well as the protection of stone levees by cylinders.

Low groynes were also built of wire cylinders. They are mostly used where the river bed consists of cobbles and gravel. Where the river bed consisted of sand, the permeable type of pile hurdles and light type of mattress were employed. Concrete blocks were also adopted for construction of low groynes.

Wire cylinders have many advantages over other types such as (*a*) flexibility of protection, (*b*) ease of handling and construction, (*c*) ease of repair and replacement, and (*d*) cheapness; but they have one important defect, that they are short lived, as galvanized wire cannot last for more than 10 to 20 years. Just after the war, wire was so scarce and costly that much of the work in Taiwan had to be done using the expensive and uncertain monolithic concrete construction.

1. Hsu S. T., "Spur and Permeable Spurs of the Yungting River", *Chinese Institute of Hydraulic Engineers, Monthly*, vol. 11, 1936 (Chinese), p. 414.

Bank protection in Japan

The swift current rolling down the steep rivers of Japan menaced the embankments and banks for centuries and methods of bank protection have been developed since the eighth century. Later on Dutch engineers introduced the use of mattress works in Japan. At present, quite a variety of bank protection works are used, including groynes. A list of these works is given below:

(*a*) *Bank heads*
- (*i*) Pile cribs filled with stone
- (*ii*) Earth protection by stone and piles
- (*iii*) Wire cylinders

(*b*) *Groynes*
- (*i*) Solid type
  - a. Rubble mounds
  - b. Wire cylinders
  - c. Paved stone with or without sand and gravel core protected with piles on both sides
- (*ii*) Permeable type
  - a. Hurdles of timber or reinforced concrete piles
  - b. Piles with mattress and wire cylinder foundation
  - c. Piles with planted bamboo branches
  - d. Piles and hangers
  - e. Piled roof and rafter type (net-like)
  - f. Tetrahedron and cribs (see below)

(*c*) *Bank revetment*
- (*i*) Slope protection
  - a. Cylinders
    - i. "Snake-like" cylinders (sausages) of woven bamboo or wire mesh
    - ii. Timber cylinders
    - iii. Willow cylinders with the aim of growing willow bushes
  - b. Fence and sheet piling
    - i. Pile fences with bamboo
    - ii. Pile and board
    - iii. Pile and mattress
    - iv. Sheet piling
  - c. Pitching or paving
    - i. Stone pitching
    - ii. Concrete pavement
    - iii. Cobble or stone pitching in frame of iron concrete beams

(*ii*) Toe protection

a. Mattresses
   i. Lumber mattresses
   ii. Flexible lumber mattresses
   iii. Reinforced concrete mattresses, wire connected concrete or stone mattresses
   iv. Rod connected concrete block mattresses
   v. Wire mesh mattresses

b. Concrete breast walls

c. Closely driven piles

d. Sheet piles

e. Tetrahedrons, pyramids and prisms
   i. Three-post simple pyramid
   ii. Six-post tetrahedrons ("birds' claw")
   iii. Wood pole tetrahedrons ("sacred cows")
   iv. Reinforced concrete tetrahedrons (concrete "sacred cows")
   v. Square based pyramids ("diamond cows")
   vi. Roof spur ("stabled cows")
   vii. Parallel trusses ("sheet cows")

f. Cribs
   i. Triangle prism cribs (inter twined fingers cribs
   ii. Cubic cribs
   iii. Wedge-shaped cribs
   iv. Frustrum-shaped cribs.

In Japan, as in other places where current of rivers is swift and cobbles and stone can be obtained cheaply, the merits of wire mesh cylinders and wire mesh mattresses have been generally recognized. Except for cylinders under sea water, galvanized wire may last more than ten years and lasts longer in clear water than under muddy water or at places where frequent changes of wet and dry prevail. Where clay deposits are likely to occur, a layer of sand and gravel covering the cylinders may enable them to last longer. For temporary works, bamboo or willow cylinders are much cheaper, especially willow which may grow into bushes at the rate of about 1 m per year. These bushes must be cut once every two or three years, otherwise they will grow into trees and lose their ability to resist erosion.

Japanese engineers praise most the use of permeable pile groynes for deflecting currents away from banks. Groynes are placed either perpendicular to the bank or inclined upstream at an angle of about 15 degrees. The height of groynes is limited to that level which is high enough to induce deposition, and no more. Different types of groynes are used according to the slope of the river, as summarized in table 6:

Table 6

Slope of river	Type of groyne
$\frac{1}{5,000}$ to $\frac{1}{10,000}$	Pile groynes with fencing or curtains
$\frac{1}{2,000}$ to $\frac{1}{5,000}$	Pile groynes with heavy mattresses
$\frac{1}{1,000}$ to $\frac{1}{2,000}$	"Sacred cows", prism cribs, etc., with mattresses or wire cylinders for foundation

"Cows" and cribs

The various kinds of "cows" and "cribs" listed above are of Japanese origin, although tetrahedrons have also been used in China and America for river works. They are used both for toe protection and for groynes. "Cows" are so named because the side posts tied together project above like a pair of horns. They are usually built of timber, but lately reinforced concrete posts have also been much in use. The so-called "sacred cow" is composed of three parallel pairs of inclined and crossed struts, with one pair shorter than the others in front, connected by a single slanting purlin placed at the top and two stringers at the bottom bound together with struts. Besides the main members, there is a middle strut between the front pair of inclined struts, and a buried cross beam at their foot. The stringers at the bottom are also connected by cross beams in each span. The front pair of struts is buried a little under ground, and the whole tetrahedron is weighed down by wire cylinders put over the bottom stringers. The tail of the "cow", formed by top purlin and bottom stringers twined together, is also weighed with wire cylinders. Some "sacred cows" have a middle strut for each pair of inclined posts while some large ones have four pairs of struts instead of two. The size of "sacred cows" is rated according to the length of the top purlin. The largest "sacred cow" has a purlin 13 m long (46 Shaku or Japanese feet). The purlin is of a round timber with its smaller end from 21 cm (7 Japanese inches) to 45 cm (1.50 Shaku) in diameter. Other members are made of smaller chestnut poles. Posts and beams are all bound together with ropes. Two bamboo strips are put under the purlin to ensure tightness of binding.

"Sacred cows" of reinforced concrete posts and beams are of the same construction except that two connecting ties are sometimes added between the top purlin and the bottom stringers. When only two struts and a stringer form a tetrahedron with three base beams, the structure is called a "bird's claw." If four struts are bound together at the top and spread to form a square connected with cross beams at the bottom and fence posts on the four sides, it is called a "diamond cow." Both "sacred cows" and "diamond cows" are limited in length by the top purlin. They are placed along the toe of an embankment in rows like fencing with the front facing the current, or in a series of single, double or more rows to form a permeable groyne. Special types of "cows" called "stabled cows" and "sheet cows" resemble more or less triangular prism cribs and form a permeable groyne of some length. Instead of filling the space between the parallel struts with stone as in cribs, wire cylinders of suitable length are either hung over the triangular roof-like structure or placed above the floor beams and stringers.

Figures 26-29 show the typical construction of "sacred cows", "stabled cows", and "diamond cows". The arrangement of "cows" in rows and as a groyne are also shown.

A type of construction like "cows" has the merit of replacing pile type of permeable groynes and also pile fencing. The disadvantages are their lightnes sand sparseness. Because they are light, "cows" are easily dislodged or overturned by a strong current. Because they are scattered, they are not well adapted to catch silt as quickly and evenly as permeable groynes of the pile type usually are. The lightness of "cows" can be avoided by fencing around the main posts with vertical or horizontal sticks and filling the fenced space with stone. They can also be constructed to form a continuous structure instead of detached units like "cows". The rectangular type of cribs can also be used for considerable heights by adding one or more tiers of smaller cribs over the bottom crib. Figure 30 shows a triangular crib with vertical fencing, and figure 31 shows another with horizontal railing. They can be constructed either of timber, bamboo, or reinforced concrete posts. Figure 32 shows a crib in the form of a frustum of a square pyramid, which is considered to be the strongest type of crib construction. Excessive scour at the heads and sides of such groynes is very destructive to this type of structure and the whole groyne is liable to be washed away bodily.

Bank protection in India and Pakistan

Besides the protection of guide banks and their slopes and the falling aprons mentioned before, various methods of constructing groynes and bank revetments are practised in India and Pakistan.

Although hurdle groynes were not successful on the Indus at Dera Ghazi Khan, and the easy decay of wood members of the hurdles as emphasized by Spring made them impotent to defend the bank, yet the like failure of Denehy's groynes at the same site did not hinder their adoption at other places. The use of hurdles for inducing sediment deposition and thus protecting the adjacent banks has been practised quite extensively in India. In fact, as mentioned by Spring, Indians used hurdles probably a thousand years before those built on the Mississippi. In Bengal, the so-called "bandals"[1] are built of bamboos up to 50 to 60 ft in length and from 1 to 3 in in diameter and are fastened at the top by cheap string. After the receding of water in a river when the flow is divided in several channels, "bandals" are put on the shallower channels to slacken the flow and to induce silt deposition, so that the flow may be concentrated in a single channel of greater depth. Some cheap permeable groynes 40 to 60 ft long and spaced at 100 ft to 500 ft intervals have been in use on the Great Gandak (Naraini) river in Bihar. These groynes are built of sal "bullahs" (logs) and bamboos and filled with brushwood and stone. They are inclined downstream at about 45 degrees with the bank. Although the percentage of incidents or wash-out is as great as 60 per cent, their effect on bank protection, however, is appreciable in 80 per cent of cases. Hurdle type of groynes composed of two rows of piles meeting at the top and braced by cross-pieces in both directions sidewise and crosswise, have been used successfully in hilly regions. A marvellous way of sinking piles for construction of groynes on the Cosaye river in West Bengal may be mentioned here. Groynes are built with two rows of piles spaced 8 ft (2.4 m) apart and fastened together with bamboo waling pieces. A bamboo mat is placed inside of each row and the span is finally filled with brushwood. The piles are sunk in the following way. A manila rope is fastened at the top of the pile, long enough to provide two pulling lines. Where the river is shallow, men wade to the spot and erect the pole which first sinks by its own weight. Then with three men at each rope, they pull and swing the pole which gradually sinks until the required depth is reached. Where the river is deep, the poles can be carried to the spot in a boat, along with the men to pull and swing the ropes. A bamboo cross beam is fastened to the pile with two or three men standing on the beam. These men shake the pile, simultaneously pulling and swinging the ropes. Piles can be sunk in this way to a depth of 5 ft (1.5 m) or more. Four groynes aggregating a total length of 610 ft (183 m) and also a revetment of piles and brushwood, 368 ft (110.4 m) in length, constructed in this way cost only Rs3,000.[2]

1. See sub-section on bandalling.
2. Anita, K. F., "Temporary River Training, an Example", *in The Indian and Eastern Engineer*, vol. 28, No. 6, June 1936, Calcutta.

For bank revetment, bricks are used extensively along the Lower Ganga, Godavery, Krishna and Hooghly where stone is scarce. Slopes of 1 in 2 are first graded and bricks $10 \times 5 \times 3$ in ($25 \times 12.5 \times 7.5$ cm) are used. Heavy works require a paving of one vertical course of bricks laid over a flat course making a total thickness of 13 in (32 cm). Lighter works may be only 8 in (20 cm) thick, that is one brick on edge over a flat course. A cushion of ballast is provided under the bricks. The toe is usually protected by closely driven round piles 12 ft (3.6 m) in length. On the Hooghly concrete breast walls are also in use, and asphalt sheets poured in place are under trial.

Soil cement blocks

The use of soil cement blocks as a protection against wave-wash and erosion has been successfully tested in the laboratory of the Punjab Irrigation Research Institute[1] and these have also been constructed on the Beas river near Dasuya, Punjab, India. The blocks are made about 2 ft (0.6 m) square and 8 in (20 cm) thick. A loam type of soil with the following content in clay, silt and sand is found suitable for this work:

Clay	8 to 15%
Silt	12 to 25%
Sand	60 to 80%

Five per cent cement or a ratio of soil: cement = 19:1 is mixed with the soil. Results of tensile and compressive strength tests are shown in table 7. The water added for making test specimens was equal to the optimum moisture for maximum compaction, and specimens were cured under a wet coarse cloth.

Table 7

Specimen number	Curing period (days)	Tensile strength (lb/sq in)	Compressive strength (lb/sq in)
1	1	9.1	
2	3	26.2	217.6
3	7	38.4	250.4
4	14	66.2	266.0
5	28	87.6	280.0 (maximum recorded on small machine)

The following specifications are proposed by the Institute for these blocks:

(*a*) The soil should be well graded to give a high compacted density of about 2.0 and the dry-bulk density *in situ* should not be less than 90 per cent of the value obtained by the compaction method employed in the laboratory;

(*b*) The soil should have a pH value below 9.5;

(*c*) The total soluble salt content of the soil should not be more than 0.2 per cent, but sodium sulphate content should in no case be more than 0.05 per cent;

(*d*) The sodium sulphate content of the water used for compaction should in no case be more than 20 parts per 100,000;

(*e*) The soil should be free from organic matter;

(*f*) The soil cement blocks should develop a crushing strength of at least 250 lb/sq in when cured for seven days;

(*g*) The soil cement blocks should be capable of resisting 12 cycles of wetting and drying;

(*h*) The plasticity index of the soils should be between 10 to 12. But this value depends upon the structure and the chemical composition of the soil. In certain cases when the soil is well graded and obeys other specifications, the lower value will do.

1. East Punjab Irrigation Research Institute: "Soil Cement Blocks as Protection against Wave-wash and Erosion", *Research publications*, vol. 1, No. 1, Amritsar, 1949.

Bank protection in the Philippines

In the Philippines, both groynes and revetments are used for bank protection. The use of hurdles on sandy rivers has been very successful, except that sometimes the accretion of sediment at one bank causes rapid caving of the opposite bank. Solid groynes for protection of embankments are also used but the destruction of groyne heads and of their side slopes upstream seems too great. The use of wire cylinders and wire netting is not so extensive as in other regions for swift mountain rivers. The breaking of wires results in disintegration of the whole piece into loose cobbles. Perhaps wires used there are too thin or perhaps the river water contains some special chemicals destructive to wires. Wire cylinders should not rest on any rigid structure such as on the top of steel sheet piles, because the bending of the cylinder over hard metal might break the wire.

The merits of steel sheet piles and the defects of wooden sheet piles as toe protection are fully revealed in sharp contrast on rivers in the Philippines. Wooden sheet piles cannot be made water-tight easily and unless great care is taken, the leakage of fine particles of soil through the piles may eventually lead to the collapse of the paving along the slope.

COMPARISON OF MERITS AND COSTS OF BANK PROTECTION WORKS

Bank protection, owing to the extensive nature of the work, is always bulky and expensive. Unless the property or the land to be protected is extremely valuable, such as the water front of an industrial city, engineers as well as tax payers are prone to choose the cheapest type rather than a stronger but expensive one. Moreover, there is yet no such criterion as a factor of safety for use in designing structures that can be used for bank protection, and cost becomes the dominating factor in choosing the type. All examples cited in this and the next chapter are developed by engineers with the aim of achieving the protection of banks by the cheapest ways and means. In fact, many failures of bank protection work may be attributed to over-economizing in construction.

Therefore, in discussing the merits of different kinds or methods of bank protection, it is a matter of comparing the costs of the different methods and types that can fulfill the task. The cost should include the initial outlay as well as the annual maintenance. As the cost of material and labour varies so much from country to country and furthermore certain material which is abundant in some places may not be available elsewhere, it is impossible to make a numerical comparison of the costs of different types of work, to say nothing of the highly varied conditions of rivers in different countries where bank protection works are carried out. In the following paragraphs, an attempt is made to indicate only the general preference of one method over another, in the light of experiences gathered within as well as outside this region.

(1) When river banks are subject to erosion, it is highly desirable that protective measures be taken as early as possible rather than to tackle the problem when erosion has become serious. The cost of bank protection, if deep scour has already developed and the river channel has formed sharp and irregular bends, will be many times more expensive than if the problem had been tackled in its initial stage. On some occasions the river, after showing signs of erosion for a short or long period without causing serious scour, may change the direction of attack to some other place; while on other occasions erosion may be persistent and may develop serious trouble. It is for the engineer to make a careful study of the changes that have occurred and use his knowledge and judgment to decide whether action should be taken or not.

(2) If a deep scour has already developed, which usually attracts the current toward it, particularly in large rivers, it is preferable to divert or deflect the attacking current somewhere upstream rather than to protect the place of scour directly. Bank heads, repelling groynes or Denehy's groynes are examples of such constructions and on occasions some dredging of the shoals upstream may be necessary with a view to changing the direction of the attacking current. It is true that such structures, if not properly designed, may induce adverse effects such as the over-deflection of the curent and the consequent erosion of the opposite bank. The lay-out of such structures should, therefore, be tested in models before actual work is commenced.

(3) Regarding the relative merits of continuous and non-continuous protection, experience seems to show that their use depends on the sediment load (bed and suspended load) of the river and on whether the river is of the aggrading or of the meandering type. For non-continuous protection such as a series of groynes, be they of the solid or permeable type, or as a series of hurdles, best results would be obtained if sediment could accumulate between the groynes and a permanent foreshore could thus be developed. For solid groynes, the accumulation of sediment deposits would enable the gradual raising of the height of groynes in stages over the deposit between them, thus greatly reducing the cost. Therefore, the use of non-continuous protection is most suitable for the aggrading type of river where sediment load is relatively high in respect to the carying capacity of the river. The usual wide and shallow cross-sections of aggrading rivers are particularly adapted to the use of permeable construction in which piles or posts of ordinary length can serve the purpose. The successful use of such structures in aggrading rivers is demonstrated by the construction of "bandals" in deltaic rivers in Bengal (India), of bamboo fences in rivers of the Pyuntaza Plain in Burma and of permeable groynes in the Yungting river in China. Rivers adjacent to the hills, carrying a large amount of shingle, such as rivers in Japan and New Zealand can also be considered as aggrading rivers. Hurdles have been successfully employed in these rivers.

(4) For the meandering type of rivers, where the water course is well defined, deep and winding, like the Irrawady and Yangtze, it appears that continuous protection by revetment is preferable to non-continuous protection. On account of the great depths encountered at the concave banks where protection is deemed necessary, the use of non-continuous protection such as bank heads or short groynes, which have to be spaced quite closely and have to be brought up to a certain height in order to make them effective, is much too expensive. Revetment has the advantage of achieving the object of protection without adverse effect on the flow in other parts of the river and does not create any undesirable obstacles to navigation as groynes may do, especially the low groynes when submerged at higher stages of flow.

(5) With regard to the suitability of material for bank protection work, keeping both the availability and the cost of material in view, it appears that stone and brushwood are the most practical ones to be recommended. Brushwood includes any untrimmed scrub, bamboo, willow, "pilchhi," etc. Concrete and asphalt even if available are generally too expensive to be considered for use in this region. If boulders or cobbles are available at site, the use of wire mesh as casing, in the form of wire mesh cylinders or wire mesh mattress, is practical. Recent experiments on the use of bricks burned at site, or cement stabilized soil blocks are encouraging.

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Chapter IV

RIVER WORK PRACTICE IN OTHER REGIONS

RIVER WORKS ON THE MISSISSIPPI AND OTHER RIVERS IN THE UNITED STATES OF AMERICA

Background

The Mississippi is one of the largest rivers in the world. Its drainage basin is exceeded in size only by those of the Amazon and the Congo rivers, its total area being 3,200,000 km² (1,240,000 mi²), as compared with 7,000,000 km² (2,722,000 mi²) for the Amazon and 3,660,000 km² (1,425,000 mi²) for the Congo. The total length, from the source of the Missouri to the mouth of the Mississipi, is 6,750 km (4,194 mi), and surpasses that of the Amazon. The maximum discharge on record, 75,600 m³/sec (2,662,000 ft³/sec) at Arkansas City in 1927, is only next to that of the Yangtze river, which has a record of 80,180 m³/sec (2,830,000 ft³/sec) at Tatung in 1941.

As to the utilization and control of its waters, works on the Mississippi and its tributaries must rank at the top in history. Not only does its 2,560 km (1,600 mi) of levees below Cairo, Ill. comprise one of the most extensive flood protection systems, but also over 8,000 km (5,000 mi) of inland waterways, 1.8 m to 4.5 m (6 to 15 ft) deep, form a unique system of navigation for a single river. River training as well as bank protection, therefore, form an indispensable part of its works and owing to its long history of attempts to improve the methods of construction, practice on this river system is well worth a study by workers on other large rivers. Owing to different sizes of the main river and its tributaries, it is necessary to study the records of each section, the Upper Mississippi, the Missouri, the Middle Mississippi and the Lower Mississippi, separately. Table 8 has been compiled from the data given by Jackson.[1]

Works on the Upper Mississippi

The regulation of the Upper Mississippi was initiated about 1878, mostly by bank protection, for the purpose of improving its navigation channel below Minneapolis. Fascine mattresses were used almost exclusively for revetment, extending in general from as near the mean water surface as practicable, the highest limiting level being about 1.3 m above that stage, to a distance 1.5 m from the toe of the bank into the channel. The average width of the mattress was about 8 m (26 ft). The bank was graded to a slope of from 1 in 2 to 1 in 3. A portion about 2.5 m to 3.0 m was paved with riprap stones, the thickness of which varied from 38 cm at and near the lower edge of the mattress, and tapered to 15 cm at the upper limit of paving. No provision was made for drainage of the bank.

The average cost of this work was about $18.00 per metre of bank and about 500 km of bank were scheduled for protection. Total expenditure from 1878 to 1931 was about $7.5 million for construction and about $500,000 for maintenance. The river was then improved by canalization for a 9 ft (2.8 m) deep waterway completed in 1940. About 430 km of bank protection works are still effective.

Works in the Missouri and the Middle Mississippi

Works on the Missouri and the Middle Mississippi may be treated as one unit as they are of the same type and for the same purpose, namely improvement of waterway by regulation. A 1.8 m (6 ft) waterway was aimed at since 1878, but after the construction of the Fort Peck reservoir, the channel is now regulated for a 2.8 m (9 ft) waterway. The Middle Mississippi is improved for a 2.8 m (9 ft) channel, but a 3.65 m (12 ft) channel is expected to be approved.

The regulation work consists of two main types, the permeable groynes and the bank revetment; the former is mainly aimed at the correction of the width and the curvature of the river, the latter for the stabilization of caving banks. No correction of the bed is attempted as in European practice. Dredging is always incorporated as a part of the improvement works to help and hasten the efficient working of regulation as well as to keep the low water channel free from deposits due to local changes

1. Jackson, T. H. *Bank Protection*, Report No. 34 to the XVIth International Congress of Navigation (Brussels, 1935), 1st section, 1st communication.

Table 8

Physical data of the Mississippi river system

Section		Upper Mississippi	Missouri	Middle Mississippi	Lower Mississippi
Location		From St. Paul to mouth of Missouri	From Sioux City to mouth	From mouth of Missouri to mouth of Ohio	From mouth of Ohio to Gulf of Mexico
Length (km.)		1,035	1,230	313	1,732
Average L. W. slope		0.000 086	0.000 157	0.000 114	0.000 0664 Cairo-Angola 0.000 0076 Angola-N. Orleans
With of river at bankful stage (m)	Min. Max. Av.	60 to 340 1,030 to 1,550 430 to 1,000	270 2,570 970	430 2,190 1,060	635 2,380 1,480
Depths (m)	Between MLW & MHW	3.0 to 6.1	3.0 to 4.9	7.6 to 12.1	11.9 Cairo-Angola 4.6 New Orleans 1.2 Gulf
Depths (m)	Bank above MLW	4.3 to 5.5	3.0 to 3.7 (Kansas City) 4.9 to 6.1 (Below K.C.)	6.7 to 11.0	13.4 Above Angola 8.6 Angola-N. Orleans 1.2 at head of pass
Depths (m)	In sharp bends at bankful stage	4.6—6.1 to 6.1—12.2	6.1 to 9.3	12.1 to 22.9	30 (upper 1/3) 33.2 (middle 1/3) 45.7 (lower 1/3)
Velocity at curves during high water period where erosion is heavy (m/sec)		1.8 to 3.0	2.1	2.1 to 3.0 (Caving occurs at 1.5)	2.1—2.7 Heavy 2.7—3.9 Severe
Sediment load (ppm)		200	5,000 (Kansas City) 20,000-50,000 Maximum	5,000	553
Material	Bed Bank	Sand San to clay, loam on top	Sand Fine sand and loam	Sand 70% sand, 20% silt, 10% clay	Alluvial, mostly sand Alluvial
Discharge (m³/sec)	MLW MHW Max	187 to 565 900 to 4,960 3,400 to 9,950	227 to 565 3,380 to 7,350 5,080 to 25,500	1,700 (St. Louis) 14,080 36,700	3,396 to 2,960 32,300 to 37,200 57,000 to 65,000
Average annual duration of banful stage (days)		18 to 40	0.5 to 4.0	4 to 36	23 at Cairo to 108 at New Orleans
Annual rate of caving (m/year)		0.6-1.0 Moderate 1.5 Heavy 3.0 Severe	60-110 Moderate 150 Severe (Recorded maximum 400 per month)	6 to 15 Moderate 46 to 152 Heavy 1,390 Maximum	6 to 22 Moderate 61 Heavy 183 Severe
Radius of bends (m)	Moderate Steep	1,220 to 3,040 306 to 1,830	1,830 610 to 1,220	3,660 1,830	3,050 to 49,000 1,830 to 3,050

of sediment and velocity. The estimated cost of new work for this navigation project of the Missouri is $312 million with a yearly maintenance charge of $5 million.

Permeable groynes and closing dikes

A single type of permeable groyne was used with great success for the river training works on the Missouri and the Middle Mississippi.[1]

These groynes consist of rows of piles in clumps, spaced 4.5—5.5 m (15—18 ft) apart. Each clump consists normally of three untreated wood piles driven obliquely to a depth of 6—9 m (20—30 ft) into the river bed and tied together at the top by wire cables to form a tripod above the bed. The clumps in each row are staggered with clumps in adjacent rows and are lashed together with stringers, thus binding them together as a unit. The foundation of the groynes is protected from scour along the foot of piles by woven willows or lumber mattress sunk in position by weighing with ballast stone before the piles are driven. These mattresses extend 7.6 m (25 ft) upstream and 15.8 m (52 ft) downstream of the centre line. They also extend 13.7 m (45 ft) beyond the head of the groyne. These pile groynes are built out from the river's high bank which is protected by grading and paving against any chance of being outflanked. The groynes extend to the river in a slight downstream direction and to the limit lines determined for training of the section. Two rows are usually sufficient to form a groynes, but three to five rows together are not uncommon, depending upon local conditions. A curtain of tree branches is sometimes hung from the piles to expedite sediment deposit by the river. Figure 33 shows a plan and profile of a two-row groyne with a three-row head. Modifications of this basic form of design are also in use for special purposes.

These groynes are more desirable to be located on the convex side of a bend rather than on the concave side, if the training work is to correct a wide and shallow reach and confine it in a bend of most suitable curvature determined by studying the natural bends and cross-sections of the river. It is also very desirable to avoid placing groynes on both banks of the river and opposing each other, as this frequently leads to the formation of a bar in the middle of the river. The construction of groynes on the convex bank would start the erosion of the opposite bank which can be stopped and held by revetment works when the limit line is reached.

To build a system of groynes, it is necessary to start from a point where the river is stable and progress downstream. The first groyne in a system is frequently angled considerably downstream and is placed nearly parallel to the final position of the talweg. This is for the purpose of directing the flow of the stream, as it leaves one bank towards the opposite concave bank. This not only helps to maintain the channel crossing between banks, but tends to reduce the velocity of flow through the groynes.

This permeable type of construction is also used to close chutes and secondary channels, but it is not adequate to build merely a closing dike directly across the head of a chute. In fact, a single permeable closing dike across a chute will be partly destroyed—frequently a portion near one of the ends—due to the increased velocity through the structure, when the river rises and the head against it increases, especially when there is debris lodged behind the structure. To avoid this destruction, it is necessary to start upstream from the chute, preferably at a point where there is a curvature in the natural bank which, prolonged, passes in front of the chute. Groynes are then built out from the high bank with their thin outer ends terminating at the line of the curve and the current of the stream is thus deflected away from the head of the chute. The bedload, which tends to move towards lower velocities, thus moves into the chute and together with its silt in suspension, builds up land behind the closing dike and effectively close the chute.

In a modified form, this type of construction is also used to protect banks which are too low or are too unstable for ordinary revetment and to fill up irregularities in a rugged bank line; such as training walls to guide the current or contract the stream channel at crossings; or as baffle dikes to induce filling behind training walls. At some places the existence of stone in the river bed may reduce the penetration of piles to such an extent that the groyne becomes insecure. In such cases, a loose rock, crib, or braced clump groyne is constructed.

Woven willow mattress, lumber mattress and fascine mattress

Bank protection works on the Missouri and the Middle Mississippi consist largely of revetments along the concave side of bends.[2] The revetment comprises mattress work below low water line and grading and riprapping above that line, with the following dimensions (see figures 34 and 35):

(*a*) Extension of mattress channelward beyond the toe of the bank by about 15 m (50 ft) and landward to a point from 0-1 m (0-3 ft) above the mean low water level for the Missouri and from 0-1.5 m (0-5 ft) for the Middle Mississippi. A layer of spalls is provided under the paving on the Middle Mississippi.

1. Corps of Engineers, US Army, *Improvement of the Missouri River*, third revised edition 1914, Kansas City, Mo., USA.

2. Jackson, T. H., *op. cit.*

(*b*) Average width of mattress proper: about 25 m (82 ft) for the Missouri and 30 m (100 ft) for the Middle Mississippi.

(c) Lap of adjoining mattresses 9 m (30 ft).

(*d*) The paving extends landward to within 0.3 m (1 ft) of top of bank or to an upper limit of 5 m (16 ft) above mean low water level on the Missouri and 6 m (20 ft) on the Middle Mississippi. Average width of paving is 14 m (45 ft) and 18 m (60 ft) respectively on a graded slope of 1 in 3, and the thickness is 30 and 35 cm (12 and 14 in) respectively.

Three types of mattresses have been used, viz., the willow or brush mattress, the fascine mattress and the lumber mattress, the last named being used most extensively since 1929 on the Missouri (about 65 per cent) and since 1920 on the Mississippi (95 per cent).

(*a*) *The woven willow mattress* (figures 36 and 37)

The Missouri river type of mattress is commonly constructed of brush using basket weave strengthened by longitudinal and transverse cables with a heavy header called the wisp (see figure 36). It is cribbed along the upstream or outer edge to hold ballast stone, and is anchored to the shore by piling or by "deadmen" (anchor of wood). The entire mattress is sunk into place and ballasted down by a layer of one-man stone and in addition a compact 0.3 m (1 ft) layer of stone 3 m (10 ft) wide is used for the length of the shore edge to tie the mattress into the lower edge of the bank paving. Material used per m^2 is about 0.25 m^3 of brush and 0.083 m^3 of stone (3.4 cu yd brush and 1 cu yd stone per 100 sq ft). The Mississippi river type of this mattress is constructed of brush, woven through poles spaced at definite intervals (figure 37), and is also strengthened by longitudinal and transverse cables. Cribs to hold the ballast stones in place are constructed for a width of from 3.7-11.0 m (12-36 ft) along the edge of the mattress and a comppact layer of stone is also used along the shoreward edge to tie the mattress into the bank paving. Material used for constructed is much the same as that mentioned earlier for the Missouri type.

(*b*) *The lumber mattress*

This mattress is a development of the woven willow type and is used where lumber may be obtained at a relatively low cost. Planks 2.5 × 10 cm (1 × 4 in) are used throughout. Separate boards are held in place by weavers spaced about 1.2 m (4 ft) apart, connected by nailing and by wiring. Two rows of cribs 2.5 m (8 ft) square and 0.3 m (1 ft) deep are constructed along the streamward edge of the mattress to hold the ballast stone in place. Sinking is accomplished as with woven brush mattress by casting one-man stone from barges, starting at the upstream inshore corner. About 0.029 m^3 of lumber and 0.083 m^3 of stone is required per m^2 of mattress (111 F.B.M. of lumber and 1 cu yd of stone per 100 sq ft). The construction is shown in figure 38.

(*c*) *The fascine mattress*

The fascine mattress used on the Upper Mississippi consists of a series of cylindrical bundles of willows or fascines bound tightly together by binding poles spaced as shown in figure 39. It forms a mat about 50 cm (20 in) in thickness and is anchored in place by stone. Materials required for construction are about 0.36 m^3 of brush and 0.25 m^3 of stone per m^2 (4.4 cu yd of brush and 3.0 cu yd of stone per 100 sq ft).

The fascine mattress on the lower river is heavier than that mentioned above (figure 40). Heavy cables are used to hold the fascines together with crib construction to hold ballast stone. Total thickness comes to about 76 cm (30 in). The mattress is anchored to the top of the bank by "deadmen". Materials required per m^2 are: 0.58 m^3 brush and 105 kg stone (7.0 cu yd of brush and 1.2 t of ballast stone per 100 sq ft).

Cost of bank revetment was as follows:

Missouri (1929-35)—mattress \$1.38 per m^2 (\$12.29 per 100 sq ft), paving \$2.36 per m^2 (\$22.05 per 100 sq ft), total cost \$65.6 per lineal metre (\$20.00 per ft) of the bank. Mississippi—(up to 1928) \$43.60 per metre (13.30 per foot), and (1929-1933) \$74.0 per metre (\$22.50 per ft) of the bank.

Bank protection in the Lower Mississippi

Work on the Lower Mississippi has two main purposes: to provide a regulated channel for navigation and to achieve flood control. The flood control rests mainly on the levee system along its banks, although the use of reservoirs, flood-ways etc., are also projected. River regulation and bank protection are therefore essential to supplement the two main purposes. Various means and methods had been tried on this river, including hurdle type of groynes etc., but without much success. Finally the improvement of navigation channel is practically dependent upon dredging as the sole solution. The effect of bank stabilization by revetment on a navigable channel had been early recognized. Later on, a laboratory study on the meandering of alluvial rivers carried out at the US Waterways Experiment

Station, Vicksburg, Miss., by J. F. Friedkin in 1942-44,[1] indicated that bank stabilization is beneficial to the low water channel. Also, for the purpose of shortening the course of the river and of hastening the flood flow to the sea, cut-offs had been carried out which were found to be a source of menace to the upstream banks. A scheme of large-scale bank revetment was therefore formulated in 1944 for a section of the river between Cairo and Baton Rouge, a total distance of 1,170 km (737 mi); 44.5 per cent or 527 km (327.5 mi) of effective bank revetment was already in place at that time. The cost of revetment was estimated at about \$407,000—438,000 per km (\$650,000—700,000 per mi) with a total cost of \$165 million for the new work. This is certainly the most extensive bank protection work ever carried out for a single river.

Several types of mattresses are used together with bank paving on the Mississippi and will be described in the following sections. The work generally extends from 15 to 30 m (50 to 100 ft) streamward beyond the toe of the under water slope (of the bank). The upper edge of willow mattress is placed as near the low water surface as practicable, limiting the maximum height to 6 ft above it and the edge of the articulated concrete and the asphalt mattress is placed so as to permit a good dry connection with the paving, the limiting height above mean low water being 6 ft, except when it extends as a paving. The average width of mattress varies from about 49 m (160 ft) in the upper reaches to about 129 m (425 ft) in the lower reaches, with an average of about 91 m (300 ft) for the entire Lower Mississippi. The overlap of adjoining mattresses varies for different types, being 4.5 m (15 ft) for a fascine, 1.5 to 3 m (5 to 10 ft) for an articulated concrete and 3 m (10 ft) for an asphalt mattress.

The bank is graded from about 1.5 m (5 ft) below the mean low water at a slope of 1 in 3 except in those cases where the material requires a flatter slope. The paving is either of 25 cm (10 in) riprap stone, or of a 10 cm (4 in) monolithic concrete slab or of 7.5 cm (3 in) monolithic asphalt. A layer of crushed stone spalls or gravel is placed under the paving in some districts. The average width of paving is about 27.5 m (90 ft).

Provision is made for the drainage of the area immediately behind the back.

The willow framed mattress

The willow framed mattress is a heavy type of construction used on the Lower Mississippi, usually in rectangular units of 46 $\times$ 31 m (150 $\times$ 100 ft), and 90 cm (36 in) thick. A layer of willow poles and two layers of willow brush are placed at right angles to each other and held together by a series of frames including the upper and lower chords as shown in figure 41. It is stiffened with longitudinal poles which in turn are fastened to the upper chords of the frames, forming pockets of 3 m (10 ft) square to receive the ballast stone. Stanchions or posts 80 cm (32 in long) are placed between stiffening poles at intermediate points to connect vertically all the frames and members to form a more rigid construction. The mattress is anchored to the top of the bank by "deadmen" and its buoyancy is overcome as in the case of other wooden mattresses by loading stone to it from barges. About 730 kg (1,600 lb) of stone and 4.6 m^3 (6 cu yd) of brush are required for a 3 m (10 ft) square mattress (78 kg stone and 0.49 m^3 brush per m^2). The cost per 100 sq ft was approximately \$15.50 during the period 1928-33. Heavy fascine mattress as described in sub-section "woven willow mattress etc." is also used on the Lowed Mississippi and cost about \$2.03 per m^2 in 1928-33.

The articulated concrete mattress[2]

Owing to scarcity of lumber, willow and manpower, articulated concrete mattress is the most widely employed type of revetment under water. It consists of concrete slabs 34 cm (14 in) wide, 102 cm (3′10¼″) long and 7.5 cm (3 in) thick, spaced 2.5 cm (1 in) apart. The slabs are cast in groups and are reinforced and connected by No. 8 non-corrosive wire, to form sheets of 1.2 m (4 ft) by 7.6 m (25 ft) each containing twenty slabs (see figure 42). Units are laid side by side on the inclined way of a launching barge and 8 mm (5/16″) or 11 m (7/16″) launching cables occupy the 44 mm (1¾″) spaces left between each pair of adjacent units. Loops of the reinforcing wire mesh projecting from the ends of the slabs of adjacent units are fastened to the launching cables by clips and twisted wires to form a mattress 7.6 m (25 ft) wide measured normal to the shore line and 42.7 m (140 ft) long measured parallel to the shore line. The mattress is anchored ashore by non-corrosive wires leading from the loops of reinforcing mesh projecting from the shore and slabs to screw anchors spaced 1.2—2.4 m (4—8 ft) apart. It is launched partly by withdrawing the barges riverward, concurrently "paying off" the launching cables.

As soon as the space is sufficiently clear, a second series of units is placed over them, made fast to the launching cables and attached to the riverward ends of the preceding series by means of projecting loops of the reinforcing mesh at the shore end. The procedure of launching and of adding units is repeated until the mattress has been carried as a continuous sheet a certain distance riverward to the toe of the underwater slope. The launching cables are then cut at the barge and the outer end of the mattress

1. Friedkin, J. F., *op. cit.*

2. Elliott, D. O., *The Improvement of the Lower Mississippi River for Flood Control and Navigation*, US Waterways Experiment Station, Vicksburg, Miss., 1932.

is released. The barge is moved upstream into the bank ready to assemble the next mattress overlapping 1.5 to 3 m (5 to 10 ft) on the one previously placed.

The normal requirement of material for this type of mattress is 0.073 m^3 of concrete per m^2 (0.84 cu yd per 100 sq ft) and its cost was the lowest among the types of mattress used in the 1928-33 period, being \$1.50 per m^2 as compared to \$1.63 for a willow framed mattress.

The asphalt mattress[1]

This mattress consists of reinforced strips of asphalt concrete with widths up to 66.5 m (217.5 ft). The thickness varies from 5 to 7.5 cm (2 to 3 in) and the length is determined by that of the "stringout" barges which guide the laying plant. The reinforcement consists of 7 mm (9/32 in) launching cables, spaced at 90 cm (3 ft) intervals and fastened to 1.8 m (6 ft) widths of welded fabric of No. 12 gauge wire (figure 43). Sand, loess and asphalt are dried, heated and mixed on a floating plant and the hot mix is carried to the casting and laying plant in an insulated barge. When the hot mix is conveyed by screw conveyers to a storage car of hopper type and dumped by hydraulic dumping mechanism into finishing machines, the machine distributes the mix over the deck, tamping and compressing it at the same time to form the finished mattress. The machines run on rails along the edges of the working area of the deck. Dimensions of a single launch area are 9 $\times$ 66.5 m (50 $\times$ 217.5 ft), which are the approximate dimensions of the deck area. The launch is cooled to about 125°F by means of water jets both above and below and is then laid across the deck and over a curved apron on the shore side of the barge until the rear edge is in the former position of the front edge. The operation is then repeated over and over again, care being taken to obtain a thorough bond between successive launchings by hand tamping. Reinforcing fabric and cables are drawin up from below the deck and the hot mix is kept from sticking to the deck by the use of mica flakes. Successive mattresses are laid with upstream overlaps of 3 m (10 ft) or more.

The mattress weighs about 64 kg per m^2 (13 lb per sq ft) and its strength varies from 970 to 1,940 kg per linear m (7,000—14,000 lb per ft) at 4½°C (40°F). Material required is 0.05-0.075 m^3 per m^2 (0.6—0.9 cu yd per 100 sq ft). The principal advantage of this type of mattress is its flexibility but complicated equipments are required for laying it. This type of construction is also used for upper bank paving, with a little variation in the proportion of its aggregates. Cost of asphalt mattress was about \$2.46 per m^2 in 1929-30.[2]

The construction of bank revetment works

Four distinct steps can be prescribed for the construction of revetments. They are, in the order of their performance: clearing, grading, sinking the mattress, and paving.

The clearing of the bank includes the removal of trees, brush, stumps, logs etc., both above and below water and for some distance back on top of the bank. The material cleared out from the bank should be disposed of elsewhere. The work should be performed well in advance of grading, especially where the growth is heavy and the caving severe, in order to eliminate as much as possible the occurrence of snags below the low water surface. Snags and stumps under water are difficult to be cleared and their existence is injurious to the mattress as they may hold the mattress away from the bank. Stumps with long tops are particularly objectionable since they may cause heavy stress in the mattress which may rupture it over a large area.

After clearing, the next step is to grade the bank to the required slope. The stability of the underwater protection depends largely on the extent of this grading both as to slope and limits above and below mean low water, which also depends largely upon certain conditions materially affecting caving. These are: height of bank, variation in water surface, severity of current action and the character of the material of the bank. Ditches for draining any depressions at the back of the bank are constructed in the bank along with the grading.

Mattresses are constructed and sunk after grading is completed. It is not advisable to place willow or lumber mattress too much higher than the low water level, a maximum upper limit of 1.3 to 1.8 m (4 to 6 ft) is allowed above and below the mouth of the Ohio river. Specifications for the Lower Mississippi river require that the connection between the brush mattress and regular paving should be provided by a 1.8 m (6 ft) strip of riprap varying in thickness from 50 cm (20 in) at the mattress to 25 cm (10 in) at the paving. Special requirements for this connection also are prescribed for the river above the Ohio. The width of the mattress must be determined by the possibilities of scour at the toe of the slope.

The paving of the upper bank is the final step of revetment work. Riprap is considered the best type, although because of the scarcity of stone, concrete blocks or asphalt sheets have

1. Vogel, H. D., Paper S.1-C$_2$ XVIIth International Congress of Navigation, Lisbon, 1949.

2. Barrows, H. K., *Floods, their Hydrology and Control* (McGraw-Hill Book Co., 1948), p. 207.

been used on the lower river. Also, on the lower river, paving is carried to the top of the bank, but a certain limiting height is fixed on the upper river depending on the height of the bank. Passage of local drainage over the paving also is provided in most cases.

Dumped stone for bank protection

Dumped stone is used extensively on rivers throughout the United States for protection of river banks and embankment slopes against high velocities. Criteria as outlined in subsequent paragraphs have been developed from extensive field experience and test installations.

The thickness of riprap required for bank protection is dependent on the velocity of the current within 10 ft of the bank. Velocities measured near the bank should be used rather than average velocities in determining rock requirements. The required thickness can be determined by reference to table 9.

Table 9

Current velocity (ft/sec)	Riprap thickness (in)	Max. size (lb)	Riprap grading av. size (lb)	20% size (lb)	Filter thickness (in)
6 to 12	12	150	30— 50	20	6
12 to 15	15	250	50— 80	20	6
15 to 18	18	400	80—150	20	6

In regions where hydraulic dredge tailings are readily available, a 12 in layer of cobbles with a maximum size of 10 to 12 in is satisfactory for velocities up to 8 ft/sec. Where channels are formed by banks or levees of fine sands or silts which are readily erodible, protection by a layer of gravel or crushed stone will be required for velocities between 3 and 6 ft/sec. Such a layer should be 8 to 12 in in thickness.

In most cases, riprap should be terminated in a rock toe at the level of the stream to prevent undermining the channel protection. The toe should have a minimum base width of 6 feet and a minimum thickness of 3 ft with 1 on 1.5 side slopes. Where the channel is composed of sand or silt, it is considered a better practice to carry the riprap down below the channel level to a minimum depth of 5 ft vertically and omit the rock toe. On large rivers, or tidal estuaries, having a considerable depth of flow at low water stages, the riprap need be carried down only 5 ft vertically below mean low water. In such cases the rock toe is unnecesasry. Where riprap and filters are dumped under water, the thickness of the layers should be increased 50 per cent and the specifications should be adjusted to minimize segregation in order to ensure that the minimum required thickness of well-graded material will be obtained in both riprap and filter.

RIVER WORKS ON EUROPEAN RIVERS

Rivers and river works in Europe

Although rivers in Europe are not as big in size as the Missouri and the Mississippi mentioned before, their use as waterways for navigation is as intensive as that of the Mississippi system, if not more so. They are better linked with canals and canalized sections. Improvement of rivers for navigation by regulation was practised in early times; for example, the work of regulation on the Austrian Danube dates back to 1830. Observations on the movement of solid matter by Du Buoys and river model studies for the river Garonne by Fargue in the latter part of the 19th century led to the formulation of principles and rules mentioned in chapter II. These principles and rules are still followed by present-day river engineers, with modifications here and there, made as a result of later knowledge on the characteristics of river flow. It can be seen from these rules and the practice on the Missouri that the differences in practice lie mainly in the use of sills to correct depths at bends and the narrowing of crossings to induce bed scour in Europe, while nothing of that kind is being done in America. The methods of construction in the two regions differ widely, however, owing to the different sizes of rivers encountered. Also, owing to financial as well as technical reasons, many rivers were improved first for the so-called medium water regulation and later for low water regulation.

The design of cross-section

European engineers are more particular about the designed sections of regulated rivers. A parabola is mostly assumed for a crossing, while a parabola joining the bottom of the talweg to the convex bank and another curve joining it to the slope of the concave bank, are used at the bends. Figure 44 shows cross-sections designed for the section of the Danube from km 1,879 to km 2,200 in Austria.[1] These were designed for a low water navigation channel of 2 m depth and a discharge of 600 m³/sec. A similar improvement was attempted on the river Po in Italy,[2] as shown in the parabola cross-section in figure 45. This normal section is designed for a normal discharge of 500-560 m³/sec.

1. Stalcher, L., Report 21, XVIth International Congress of Navigation. Brussels, 1935.
2. Freemon, J.R., "Flood Control on the River Po in Italy", in *Transactions of the American Society of Civil Engineers* vol. 94, 1930, p. 185.

Sections of designed channel for the Rhine between Strasbourg and Lauterbourg[1] are shown in figure 46. It was not proposed to limit the bottom at crossings, but the bottom at the apex of concave banks was to be corrected by sills as shown.

Groynes on European rivers

River training in Europe was mostly done by the construction of groynes at both banks, except where the concave bank was in line with the designed width, and the banks were then protected by revetment. Training walls were used on concave side of bends where the natural banks were farther away from the designed normal lines. As training walls were not as flexible as groynes, the correction of a section designed too narrow would mean reconstruction of the wall. Groynes would therefore be more suitable under such conditions, as only a portion of the structure need be removed or added to when any expansion or contraction of the river width becomes necessary.

In plan, groynes were invariably directed upstream. The angle between the groynes and the axis of the river varied somewhat in different localities. In Germany, for the medium water regulation in the late 19th century, the angle adopted was 70 degrees. In France, on the river Loire, it was 65 degrees on the concave bank, 75 degrees on the convex bank and 80 degrees at the inflexion.[2] In the Rhone, pairs of groynes facing each other are placed in succession with their extension from each pair of groynes intersecting the axis of the channel. The central angle sustaining the extension is 160 degrees and the bisector makes an angle with the axis of the channel, varying sinusoidally from 5 to 6 degrees at the point of maximum curvature to 0 degree at the point of inflexion. Groynes were spaced from 100 to 150 m apart along the concave banks and twice this distance along the convex banks in Germany.[3] On the Loire, with width of minor bed fixed at 150 m at the apex, and 125 m at inflexions, spacing of groynes was limited to 80 m and 125 m at the apex and inflexions respectively. Figure 47 shows the arrangement of groynes on the Loire.

Several types of solid groynes were constructed on European rivers, viz., the fascine type, the fascine and stone type, the sand and gravel mound type and the rubble mound type. They were usually constructed to a height little above the mean low water or up to medium water level and then sloped up to the bank at 1:50 to 1:200. Sometimes the slope is made as steep as 1:30, as in the case of the Weser basin in Germany where the crest of the groynes heads was lower than at other places. Side slopes vary from 1:1 to 1:3, depending on the material used. The slope of the groyne heads on the river side is made comparatively flat in Germany, varying from 1:4 or 1:5 to 1:10 to 1:20, as shown by the cross-sections of the Danube and the Rhine. Top of groynes varied from 1 m to 3.5 m in width. It may be mentioned that besides attack by currents, groynes built in these northern regions would also be subject to erosion by the impact of ice and craft. To withstand this impact, fascine work was found most suitable, where brushwood was abundant and stone expensive. These fascine type of groynes were usually built on a foundation of sunken mattress, with a cover of riprap paving. Below low water there was a riprap protection along the head. During the construction of fascine work, sand was filled in the hollow spaces while natural silting caused by water flow filled the space further up. As a consequence, the fascine work became very compact and would not be easily torn out by current or wave action. Figure 48 shows a fascine type groyne on the river Rhine.

The drawback of fascine work in the case of great depths is that the work can only be done by very experienced personnel and a very large quantity of brushwood is needed. Also the portion of the work subject to alternate drying and wetting deteriorates easily. For these reasons groynes of the sand and gravel mound type have replaced the fascine type in some river works in Germany and Austria where enough gravel was available nearby. The mounds were pitched with stone riprap on the front as well as on the side slope and also on the crest. Mattress foundation was usually provided for the head. Figure 49 shows the sand and gravel mound type of groynes used on the river Elbe in Germany, both in plan and cross-section. Rubble mound type of groyne was used in Austria on the Danube below the mouth of the Enns river for low water regulation and consisted of quarry stone averaging 35 kg in weight, discharged directly from boats. Figure 50 shows the longitudinal and cross-sectional views of groynes used in the Lower Rhine in the Netherlands.

Permeable type of groyne seems to be rather unpopular in Europe. The danger of deterioration of wooden construction by teredo and similar borers in regions of saline and brackish water also makes the use of wooden construction unpopular. On the river Po, it has been reported that the "pile retards," composed of a single row of piles and exposed to strong currents, had suffered injury and required protection by riprap for permanence. Those not exposed to strong currents have, however, endured. Permeable groynes and training walls used on the river Loire are a combination of rubble mound and pile structure as shown in figure 51 with chestnut rods to form a curtain in front of the piles.

1. Pascalon, P. and Callet, P., Report 24, XVIth International Congress of Navigation, Brussels, 1935.
2. *Ibid.*, Plate I.
3. Arp and Hirsch, Report 20, XVIth International Congress of Navigation, Brussels, 1935.

In Hungary,[1] on a part of the river Tisa where the flow is slower than that of the Danube, a sort of brush-wood tent had been used in places of great depth to replace the transverse wicker-working dikes normally used. This structure consists of two sheets of plaited brushwood, strengthened and connected with longitudinal and transverse poles and set up as a tent as shown in figure 52. The tent was set up on a raft and sunk down in a standing position by loading with earth. The retarding effect of the structure induced the deposit of sediment and the formation of banks between these groynes.

Training walls and cross dikes

Training walls are used to limit or to contract rivers where the concave banks lie outside the designed traces of the river, especially for low water regulation. If the form of the curve is well chosen, these training walls on the concave side fully guide the river. On the convex banks, however, no training walls should be used. The building of two parallel training walls had been tried in the eighties and nineties of the last century, but the result was not encouraging as the river was not much improved in this way, while much expense was required in certain places to fill dangerously deep erosion behind the work. The failure of low water training works is explained by the fact that the most effective forces which modify the cross-section of the river act at higher water levels, particularly those in the vicinity of medium water during falling stages. If the cross-sections are proportionately too large at medium water level and thus the velocity of the current too low, the contraction of low water by training walls is of no avail. The case is, however, different in tidal rivers, especially where tide water contributes a greater part of the flow.

Types of construction used for groynes can also be used for training walls but with flatter side slopes on the river side. Figure 53 shows the training walls constructed on the river Elbe, with mattress foundation and riprap pitching. The combined fascine and rubble protected type used on the river Po in Italy is shown in figure 54, the steps of construction for which are described in the next sub-section.

The first or uppermost groyne on a convex bank usually inclines along the current to form a guide bank and is actually a training wall. Also training walls built at the point of inflexion at the crossing may be followed by groynes for the contraction of the crossing. A space between the downstream end of the training wall and the groyne was provided on the river Elbe, so large as to enable the passage of small boats and to give a free exit for fish, especially for young fish seeking protection behind training walls, when wave action produced by fast sailing steamers drives them ashore.

The closing of arms of a river and concentrating the flow in a single channel is one of the most important steps in regulation. These are usually done by closing dikes.

Sills and hook groynes

Sills are used to correct depths at deep pools on the theory that the deeper the bed, the shorter would be the radius of curvature of the bend, or the sharper the curve, the deeper would be the pool. Once a pool is formed, it is very difficult to correct the bed to a proper radius. The construction of a sill or a series of sills below the designed bed line is, however, of some benefit in stabilizing the bank or the training wall. Sills are usually of rubble mound or rubble and fascine types.

When groynes are first built for medium water regulation and then extended for low water regulation, the extended parts are called head sills in Germany. Of course, a groyne may be designed with such an extension to suit the two conditions of the river, the medium and low water, as shown in figure 49 in which the headsill consists of stone riprap resting on mattress work.

In cases where groynes for medium water regulation are spaced too wide apart and additional groynes are necessary, hooked groynes as shown in figure 55 may be built. This method of construction is in between the system of training walls and that of supplementary groynes. The longitudinal part of the structure lies a certain distance behind the trace and is directed upstream. As its alignment is parallel to the current, it needs not be very strong and can be erected on a silt bed foundation. The front part is of the same construction as the head of the groyne; it acts as a good guide for the current and provides uniform depth of the talweg. This also saves the cost of the more expensive supplementary groynes and provides a good space behind for breeding fish.

Bank revetment

Bank revetments are generally in use to protect concave banks from further caving. Many types of revetment works have been devised but only a few of them would be mentioned here, as the principles encountered in the design are just the same. It should be noted that types used to protect banks from wave action of passing vessels are applicable only to canal sections of small width and would not be suitable for natural rivers under current attack.

Besides the general methods of vegetal surfacing, stone and brick pitching, rubble stone on mattresses, etc. as described before, several

1. Rohringer, A., Report 25, XVIth International Congress of Navigation, Brussels, 1935.

other practices are worth mentioning for the information of engineers in the region. A railing along reed-protected banks in order to prevent any damage by cattle is one small item that may be mentioned here. More important cases are (*a*) the construction methods of shore protection and training walls used on the river Po; (*b*) the experience on the use of wood both above and below water; (*c*) the use of filter beds; and (*d*) the experimental revetments on the Tisza embankment at Ujszeged by Julius Malina.

(*a*) The construction of shore protection works along caving banks on the river Po is shown in figure 56.[1] Piles are first driven in rows from a floating plant. They are guided by a tube hung from the pile driver A. After the piles are in place, fascine rolls or cylinders with stone are placed against the piles on the river side so as to form the footing for the new bank B. River sand dredged from shallow spots is then pumped through pipelines to the site and deposited behind the fascine cylinders against the piles. If water is deep, a single line of piles may not suffice to build up the new bank, and another row of piles may have to be driven on the newly formed bed, with fascine cylinders C put on the river side as before. A second layer of sand is then pumped to the site in the same manner as mentioned above. More layers of sand may be needed until the proper height is reached, then it is covered by fascine cylinders and paved with rubble D. Training walls are built in the same way with side slopes of 2:1 as shown in figure 54.

(*b*) From the experience of bank protection in Holland, where much timber work—including sheet piles and piles—have been used, both above and below water level, the old axiom that wood beneath ordinary water level is almost imperishable while above this level it decays rapidly, is only partly true.[2] From many years of experience, it was found that wood under water may not be imperishable, because the portion close to the water surface is destroyed by the mechanical action of the waves. Rows of stakes of fir-wood showed considerable deterioration after ten years and not much more than the heartwood remained after twenty years and the necessary tightness of protection disappeared. The same trouble is experienced with boards. But this mechanical attack stops at a quarter to half a meter beneath the ordinary water level and the lower parts of the wooden construction are found to be practically imperishable.

1. Freeman, J.R., op. cit., p. 181.
2. De Bruyn, H.C.P., and Maris, A.G., Report 39, XVIth International Congress of Navigation, Brussels, 1935, 1st Section, 1st Communication.

On the other hand, wood located a little above the normal water level does not decay so rapidly in a canal busy with traffic, as in this case the waves caused by the passing vessels moisten the wood constantly. For this reason, a wooden beam could be placed at 0.20 m above normal water level on the "Merwede-Kanal" without any danger of decay. When the waves caused by wind moisten the wood continuously, it will not decay so quickly. In other cases, as mentioned by H. Wortman, wood placed at 0.15 m above the normal water level at the Noord-Hollandsch Kanaal soon tended to decay.

(*c*) The surging of waves has a particular effect on the earth slope or toe behind bank revetment works. If the revetment work is not water-tight, the striking wave will fill up the spaces in the soil behind the revetment with water. Upon the retiring of the wave, water tends to leach out. This leaching water is likely to carry some finer soil-particles with it. This constant action of leaching may finally develop the formation of holes and hollows in the slopes and consequently, sooner or later, the revetment may settle down and collapse. If it breaks at one place, it offers a greater surface for attack by the water, and then sinks all the more. To prevent this, either a tight sublayer should be provided or a flexible construction be attempted. On shores subjected to constant attack by waves, both a flexible surface pavement and a tight substructure are generally used.

To insure practical water-tightness, perhaps the best construction is obtained by means of inter-locked steel sheet-piles at the toe, with monolithic or reinforced concrete of excellent quality on the slope. This is of course very expensive. Any construction that may not entirely exclude the flow of water but may prevent the leaching out of fine soil particles will be as good as the former. To this end, several types were tried in Holland and in Hungary. At the toe, hedges with fillings of brush-bundles or fascines, or rows of round piles with brushwood or brickbat filling behind them are relatively tight. Their resistance to the movement of water is sufficient to break the aggressive force and therefore protect the bank behind from being undermined. A construction called in Dutch "post beheiing" which is made by sawing flat two sides of each pile so that the piles can be driven more closely, is another method of securing tightness.

The filter-bed used in Hungary is a good example for protecting slopes. In more difficult cases it consists of layers of filtering medium of which the particles gradually increase in size from the bottom upwards. Such a filter allows the ground-water to flow freely, but it prevents even the smallest soil particles from being washed out. At works on the Tisza crushed stone is usually applied to a thickness of 15-20 cm (6-8 in), while on the Danube, Mura, etc., pebbles are used for the same purpose. In the harbour of Budapest a triple layer composed of coarse sand, pebbles and scabble of a total thickness of 20 cm (8 in) was placed under the revetment.

(*d*) Much information of practical value has resulted from the experimental revetments constructed on the Tisza embankment by Julius Malina in 1903 and 1907. The experiment was carried out by constructing different types of revetments resting on a 1:2 slope and aggregated to a total length of 1,320 m (4,400 ft). The revetments were "supported all along by an abutment stone-packing, edged in a 1.50 m (5 ft) broad banquette lying in the mid-water line."[1] The regular slope was produced by an adjusting fill of muddy sand, as dredged from the river bed, in order to increase the susceptibility of the revetments by using this poor material. Brick and concrete were the material mostly used for revetment, but, stone rubble, bitumen, tar, lime-sand bricks and slag concrete were also used. Altogether 16 kinds of structures in 41 "alternations" were constructed, which differed chiefly in the method of joint filling and in the size of concrete blocks. The results of this experiment were reported by Malina in 1910, and 30 years later by Laszloffy.[1] These results are summarized below.

(*i*) Destruction of revetments starts with cracks caused by settlement and frost, and finally they are undermined by wave action which sweeps out the material of the bank through the cracks. Chinks and cracks are not dangerous as long as they do not exceed 2-3 mm in size. It needs very careful work to reconstruct a cracked revetment, because the mortar filling mended in autumn will not fully adhere to the blocks and will fail in a thaw. The re-opening of cracks in spring leads to collapse of the revetment by spring floods in the manner already stated above.

(*ii*) Revetments stiffened with buttresses or made of any inflexible construction like monolithic revetments of concrete, slag concrete, concrete with wire-mesh reinforcement and with reinforced concrete supports, are all subject to the development of cracks due to unequal settlement and, therefore, may become undermined in the long run.

(*iii*) Vertical cracks always appear when a sudden change is set up in the earth bulk by thaw. The stiffer the revetment, the wider are the cracks. Wide and continuous cracks have appeared even in revetments made of large articulated concrete blocks with zigzag joints. Hair cracks occurring in articulated revetments made of small blocks like bricks laid flat follow the direction of joints, and are not a serious matter.

(*iv*) Horizontal cracks, if any, might be found after the subsidence of inundations, in places near the maximum flood level. This seems to show that such cracks are due to the settlement of the subsoil.

(*v*) Successful revetment works are shown in figures 57 to 61. Figure 57 shows concrete blocks constructed *in situ,* with wire mesh reinforcement. Continuous groves or furrows, 15-20 cm (6-8 in) wide and 8-10 cm (3-4 in) deep, were cut in the slope and filled with gravel, with small sizes at the bottom and larger sizes at the top, with the same spacing as the joints of concrete slabs (1.0 × 1.5 m) and served as a good filter medium (see figure 57). These blocks were bonded to the gravel in the filter grooves and the joints were provided with cardboard insertion. A total length of 350 m (1,170 ft) has stood in excellent condition for a period of 30 years. Figure 58 shows a type of patent revetment in which continuous R.C. supports (20 × 20 cm = 8 × 8 in) run beneath the joints, and the wire mesh reinforcement of 20 mm (0.8 in) wire with 40 mm (1.5 in) openings that lie in the middle of concrete 10 cm (4 in) thick and covers the whole scarp is fastened to them. Asphalt was used to fill the joint. All sizes of blocks varying from 1 × 1 m to 2.25 × 3.0 m were found satisfactory. In figure 59, another patent type is shown. It consists of blocks 0.8 × 1.0 × 0.1 m (2.7 × 3.3 ft × 4 in) bended to four concrete stakes 8 cm (3 in) diameter penetrating into the scarp to a depth of 0.4 m (1.6 ft). The joints are 3 to 4 mm (0.12—0.16 in) wide, filled with cement grout. This pavement, 50 m in length, has stood for thirty years at the time of Laszloffy's report of 1935. Figures 60 and 61 show brick paving that also stood, but the ribs in figure 60 had cracks along them. The

1. Laszloffy W., Report No. 36. XVIth International Congress of Navigation, Brussels, 1935, 1st Section, 1st Communication.

flat brick on a 20—30 mm (0.8—1.2 in) thin concrete layer, providing 10—20 mm (0.4—0.8 in) joints filled with mortar, proved good after many years and merely hair cracks were found (figure 61). The alternative of this paving, made of pressed lime-sand bricks, was not frost proof.

(*e*) Other types of bank revetment that may be mentioned here are:

(*i*) Brush rollers or cylinders on rivers with a pebble bottom (figure 62).

(*ii*) Square and rectangular concrete blocks for paving (figure 63).

BANK PROTECTION AND RIVER TRAINING IN AUSTRALIA AND NEW ZEALAND

New Zealand and Australian rivers

Rivers in New Zealand and Australia are not of the same character, as New Zealand is a country of islands, more or less like Japan, Taiwan and the Philippines in this region, while Australia is a sub-continent, part of which is desert land. Also, the topography and geology of New Zealand make for steep, torrential, shallow, wide and shingle-bedded rivers not so often seen in Australia. Because of the more erosive ground and greater annual precipitation in New Zealand (5,000 mm or more per annum in parts of the west coast of the South Island, and an average of 1,025 mm for the whole region; while in Australia, except Tasmania, 68.2 per cent of the land has an average annual rainfall below 500 mm and 35.9 per cent of it below 250 mm), streams in New Zealand are more numerous and have greater discharge than those in Australia. Still the two countries resemble each other in some respects, because the developed portion of Australia lies mainly at its western and southern coasts, and being bounded closely by high mountain ranges, it receives comparatively higher precipitation. Again both the regions are recently developed, and, besides the local material available, the use of modern engineering materials for river works is about the same. Thus, river works in these two regions are more or less alike and the results achieved in these two countries furnish good examples of short coastal rivers.

The control of shingle rivers

Rivers carrying shingle differ from those carrying sand in that, instead of having meandering courses, deep well-defined beds, and wide flood plains, they tend to have straighter courses and wide shallow beds with shifting and braided low flow channels, and carry most of their flood water within their beds. The shingle load of a river is derived either from the erosion of hilly country by mountain torrents or land slides, or from deepening or widening of its own bed, or from both. In New Zealand, both causes operate to a greater or lesser extent. In Victoria, Australia, the material derived from hill erosion is now mostly subsoil or sand, most of the shingle supply being derived from bed and side erosion of the stream itself. This tendency of shingle rivers to erode their banks is probably due to the tendency of the shingle to accumulate and divert the current, which in turn is due to the greater difficulty of transporting shingle compared with sand. Although the grinding action of shingle travelling down the river may tend to reduce its size and may finally reduce it to the size of sand, the river would still be unable to digest its load if the supply from the hills is unchecked and the erosion along the lower course continues. During a flood, the high velocity of the flow may be able to transport shingle downstream, but as the flood subsides the flow of shingle is checked and it piles up in heaps. The reduced velocities at half flood or in ordinary flow find it more difficult to shift these heaps than to go round them, and the channel wanders off in new directions, often attacking the banks and thus widening the bed.

The control of shingle rivers aims at assisting the river to get rid of its shingle load, so that the river bed can be confined in a narrower strip of land without shifting and further destruction of adjacent farms. Training walls may answer the purpose best but they are usually expensive. In the case of several New Zealand shingle rivers, however, embankments, or "stop banks" as these are called there, are first erected along the river to stop the flooding. A theoretical course for the river is then laid out, with defined widths and with its alignment along easy curves. Groynes of the impermeable type, with their ends curved upstream, are next built at right angles to the "stop bank" to define the course, which gradually narrows downstream to ensure the required depth at the mouth. The defined course is carefully kept clear of willows and other vegetation to prevent deposition of the shingle or obstruction of the flood flow. Outside this course, between the groynes and along the embankments, willow and poplar plantations are developed to check the flow and to assist the work of reclamation.

1. Strom, H. G., *River Control in New Zealand and Victoria*, State River and Water Supply Commission, Melbourne, 1941. Grant, A.P., "Channel Improvement in Alluvial Streams," in *Proceedings of the New Zealand Institute of Engineers*, Wellington, 1948.

Snackenberg, E.C., "Slope Discharge formula for Alluvial Streams and Rivers", in *Proceeding of the New Zealand Institute of Engineers*, vol. 37. 1951.

In a river with considerable variations between ordinary low flow, ordinary flood flow, and maximum flood flow, it is difficult to design a single channel which will meet all conditions. A cross section having double channel is therefore adopted. This is done by first laying out a regular course with sufficient capacity to take care of all floods. Works are then constructed to confine the floods in this course, with their height sufficient to prevent any reasonable risk of their being overtopped in the largest expected flood; allowance being made for any likely increase in flood levels consequent on such confinement. The portion of the original bed outside this course may be planted with trees to increase sedimentation and for reclamation later on.

Within this wide flood course, another channel is laid out of sufficient capacity to deal with the ordinary annual flood flow, which is confined to this smaller channel by works constructed at as low a level as would suffice for the purpose, and designed so as not to be wrecked by occasional overtopping during large floods. This confined flow should have sufficient velocity to keep the shingle moving but should restrain the stream from wandering. The rest of the wider course will be gradually reclaimed to a height above the ordinary floods, which will be low enough to afford extra capacity to take care of the larger floods. As the latter may not occur for several years at a time, this reclaimed land may be used for grazing purposes, but must be kept clear of trees or shrubs to maintain the flood capacity. Since the flood water that spreads over the reclaimed land in the wider course has shallower depths and lower velocities than those in the confined smaller channel, there is less tendency to scour and a thick pasture should provide sufficient protection against high floods which usually do not last long.

One important consideration to be borne in mind in designing works on shingle rivers is that if a channel is closed off suddenly by a groyne or by other means at its upper end, the lower end will be starved of shingle and will not fill up, thus remaining as a lagoon or depression, which, besides being useless for production, becomes a potential danger in case of a breach. If the grade permits, therefore, it is better to work from the lower end upwards or from the bed upwards so as to get a regular filling along the whole of the channel.

The choice and design of groynes

As in other regions, various types of permeable as well as impermeable groynes have been in use on rivers in New Zealand and Australia for river training and bank protection, but where bank protection alone is the aim, the present trend is against the use of groynes and is in favour of a continuous type of protection. Groynes suffer from many serious disadvantages: (a) maintenance is heavy, (b) their action is one of severe local resistance to the current and is often the cause of other erosion developing either upstream or on the opposite bank. It is found, therefore, that the effect of groynes on a channel, unless used as training walls, is unreliable and difficult to predict.

River conditions that affect the choice of type and design of groynes are: (*a*) fall and velocity of river, (*b*) character of bed and of material carried—shingle, sand, silt, (*c*) width of waterway and area of floodway, (*d*) depth of waterway, and height of flood rise, and (*e*) available material and funds. Groynes for the purpose of confining a river to a defined channel, especially where a double channel is required, are usually made impermeable, and earth or shingle is used for making the banks of the groyne, with a well protected head, if the groyne is located on a wide river. Permeable groynes are built on more sluggish but silt laden rivers, especially for the purpose of land reclamation. When a river carries much floating debris, however, this is apt to collect against the face of the groyne, converting it into a semi-permeable or even impermeable type.

Opinions vary as to the direction in which groynes should be built relative to the bank. The usual practice appears to be to build impermeable groynes inclined slightly upstream or at right angles to the bank, and permeable groynes inclined slightly downstream. On the Ashley river works, where a shingle river was to be controlled, impermeable groynes were inclined upstream by 10 degrees over-all. In the case of a long groyne with considerable upstream inclination and in a river with a steep slope, the natural level of water at the head of the groyne, were the groynes not there, would be appreciably higher than the natural level at the root. This increases the height of dead water backed up by the groyne, and hence the pressure on it. The same factor creates a tendency for low flow to adhere to the downstream face of the groyne. This latter difficulty may be overcome by making the groyne L-shaped, a short right-angled spur being added at the end of the main groyne towards the upstream. When groynes are inclined downstream, they should be used in groups or series, as this kind of groyne induces sedimentation only downstream of it, and a single or isolated groyne of this type tends to act as a training wall, swinging the current below it towards mid-stream with a reflex action above, which may induce the current to attack the bank there. Groynes in a group, inclined downstream, protect each other and cause a deepening of the channel along a line just outside the heads of the groynes.

Impermeable and permeable groynes

Impermeable groynes built of earth and shingle usually have a special head as shown in figure 64. Shingle-wire-mesh mattresses are laid for foundations with wire mesh crates to make the side slopes. Figure 65 shows in plan, section and profile, a small groyne entirely built of shingle and wire on the same river as that on which the groyne shown in figure 64 was built. These groyne, if constructed of fairly large stones will let a certain amount of flow through, and hence may be classed as semi-permeable. Figure 66 shows another form with fascines for foundation.

Where a fast current is eroding a river bank to a depth of about 6 or 8 ft, a useful type of groyne can be built with shingle and fascines or logs. A raft is made extending out from the bank and anchored upstream by cables. This is loaded with shingle till it sinks evenly. As it sinks, the sides and ends are built up with more logs or fascines tied to the first lot by wire, and the inside box so formed is filled with shingle until finally the raft rests at the bottom, the gaps being closed with weighted fascines or stone.

Groynes of crib type, both rectangular and triangular, built up with round timbers and filled with shingle or stone have been used on Australian creeks.

Permeable groynes built of timber piles or of old rails used as piles, either as hurdles of two rows braced together with horizontal timbers, or of a single row with waling pieces are also used.

Low level groynes and willow groynes

Two special types of groynes built in New Zealand are well worth a mention, namely the low level groynes and the willow groynes.

Low level groynes are submerged stone-mesh groynes built up strongly from the river bed, for effecting progressive and even reclamation over a large area. This system has been used on the Hutt river with considerable success to reclaim shingle beds occupying the place of eroded river flats. Stone crates, about 6 ft wide and 18 in high, are built across the shingle bed at right angles to the river flow, a stone crate apron about 6 ft wide and 6 in thick being provided downstream to prevent scour. These low weirs are spaced so that the shingle-deposit upstream of each will reach the apron of the one next above. When this occurs, another similar weir is built just upstream of the old one, using the first as an apron, and so on. Land which was bought for a few pounds per acre by the River Board when it was merely shingle, has been built up some 12 ft by means of these low groynes, and has thus become good grass land which is leased out at £A1¼ per acre.

Willow groynes are built where conditions are favourable for vigorous willow growth. Three or four rows (figure 67) of holes are first dug in the shingle bed, 8 or 9 ft apart. Each hole is filled with concrete and the resulting concrete blocks are connected with old wire cables to which willow stakes are attached. Sometimes willow stakes are fastened along a timber pile and waling groyne, like pickets on a fence. These willows, as they grow, will develop a thick belt of brush which acts as a permeable groyne. Willows should be cut or pollarded so as to encourage short growth and to keep them from growing high and unwieldy.

Continuous bank protection

Continuous bank protection may be of a temporary, semi-permanent, or permanent nature.

(*a*) Temporary protection consists of a type of construction using materials which may be expected to have only one or two years' life, such as brushwood fascines. It is used to provide temporary protection from erosion until vegetation can be established along the bank subject to erosion; sometimes it is used in conjunction with a system of groynes, to provide protection until sufficient silting has occurred between the groynes. Figure 68 shows a pile and fascine revetment, which is constructed by drawing piles along the face of an eroding bank, and packing the fascines between the piles and the bank, any space between the fascine and the bank being filled with shingle. Another method called willow pole revetment is to cover the bank with network of willows tied down with wires which are fastened to stakes driven into the face of the bank. Shingle sausages, 2 x 2 ft in cross-section and spaced at 10 ft centres are anchored with tie wire secured to driven stakes on the bank and hung over the mat to the bottom of the river. A cheaper method is to grade the bank to approximately ¾ to 1 slope, and place willow stakes vertically on the new face. Long horizontal willow stakes are placed over the vertical ones as deep as practicable from top edges and sausages of 1.5 ft diameter are secured in place and filled with shingle (figure 69).

(*b*) Semi-permanent protection comprises the use of materials which may be expected to have a useful life of a number of years, but which will inevitably fall through decay. The object of this type of protection may be to delay expenditure on permanent protection or to provide temporary protection in the hope or expectation that change in the direction of river flow may obviate the necessity for it, or to stabilize the bank pending the growth of vegetation. Timber sheet-piling and wire mesh crates filled with stone fall into the category. Shingle sausages are also in use.

(*c*) Permanent protection may consist of stone or concrete walls, or broken rock tipped from the bank to form a continuous apron. In rapid rivers of the west coast of New Zealand, huge concrete blocks have been dumped, or built *in situ*, to protect bridges. Steel sheet-piling is occasionally used for the same purpose. Sheet piling is driven near the cliff; on this a vertical concrete wall is built and tied back into the bank with iron rods. The bank above this is then graded and covered with concrete slabs.

Such permanent construction is usually too costly except in special circumstances, i.e. where some important point, such as a building or bridge abutment, has to be protected or where the water is too deep or the current is too fast for other methods to be successful.

(*d*) Two more types of bank protection may be mentioned here; the one is the rough log training wall in New Zealand and the other is the so-caleld "hen-cooping"' in Australia.

A rough log training wall consists of pyramidal trestles, built of rough logs about 6 in thick, and about 12 to 20 ft apart, with other logs wired from trestle to trestle on the side nearest the river (figure 70). Checking of the current by the log-grid causes a shingle deposit and so builds up a bank. This very much resembles the training works used in Burma.

"Hen-cooping" is a cribwork made of three lots of logs about 7 in thick, with their ends meeting so as to form a triangular tunnel, which is filled with stone. The logs may be wired or spiked together but an effective method is to bore the ends and to hold them on three steel rods, one at each apex of the triangle. This cribwork is built on a ledge cut at the base of the eroding bank, the upper part of which is battered down on to the cribwork and has green willow stakes placed on it. This structure has proved successful on the Lerderberg and the Avon rivers but is liable to be under-scoured and to collapse into the hole so formed in the river bed.

Wire mesh

Wire mesh has been used extensively on New Zealand rivers. As a rule, it is bought ready made from manufacturers. Some local bodies, however, manufacture their own. The Wainakariri Trust has made mesh on the spot by means of a small plant which is driven by a 2½ hp electric motor. This plant is capable of weaving mesh strips up to 8 ft in width. No. 8 heavy galvanized crimping wire is used, lubricated with oil to keep it from jamming. One ton of wire can make about 400 sq yd of 4 in mesh and 280 sq yd of 3 in mesh.

One engineer favours weaving wire mesh by hand, making it into nets 4 ft 6 in by 30 ft, about 1,080 ft of wire going to make up a net. One man can make two nets in a day. "Copper-bearing" wire, soft grade, "crapaud" galvanized, is used. With No. 10 gauge wire, one ton is enough for 46 nets.

The life of wire mesh work varies according to the quality of the wire and the galvanizing, and to the treatment it receives. In the fierce west coast rivers, for example, the impact of floating ice bearing stone cuts out in a few years not only the galvanizing, but the wire itself. Under normal conditions, however, a life of ten years or more can be expected.

Cut-offs

Many cut-offs have been executed in New Zealand rivers. Artificial cut-offs are preferred because they can be properly located to avoid sharp bends and the development of further meanders. The effects of cut-offs are as follows:

(*a*) Since a cut-off can be located so as to eliminate severe bank erosion by diverting the flow through a better aligned channel, bank stabilization can be more economically and effectively achieved when a cut-off is used;

(*b*) The effect on flood levels upstream depends upon the rate of deepening of the new river bed, and will extend upstream in a decreasing amount; but the elimination of losses of head along sharp bends and the increase of velocity of flood flow result in quicker propagation of the flood wave;

(*c*) Bed levels and flood levels downstream will be raised slightly but this will be very local in effect provided the downstream channel is of average section or better.

In the design of cut-offs, the alignment and the cross-section are based on the following considerations:

(*a*) Alignment of the new channel must be such as (*i*) to avoid shock losses at entrance and exit by making the cut at both ends tangential to the main direction of river flow approaching and leaving the cut, and (*ii*) to avoid the development of secondary meanders. In case of pilot cuts, the outfall of the cut is usually located slightly upstream to counter the flow from the existing meander. To promote steady development the alignment of pilot cuts is made preferably on a slight curve, the curvature being flatter than the dominant curvature of the river itself. The entrance to a pilot cut is usually made bell-mouthed, up to double section in a length of 100 to 200 m. Bell mouthing at the outlet is unnecessary as the cut develops first at the lower end and works progressively upstream.

If several sharp bends must be cut through at one place, either a single long cut or several short cuts separated by main river channels are made. A double cut, that is, a cut across the existing river course between the two ends of an S-shaped bend formation should be avoided if possible, as the flow from the upstream cut will be partly or wholly overwhelmed by flow from the existing upper bend. If such a cut is unavoidable, it will usually be necessary to defer the downstream portion of the double cut until the upstream portion is well developed. The downstream portion may then be started, but it should be located slightly downstream in conformity with the resultant flow, at the river crossing from the upper cut and bend.

(*b*) Where cuts are unlikely to develop by scour, owing to low velocities or high resistance of bed material, they should be excavated initially to the mean river section. However, the excavation of pilot cuts is sufficient in erodible material such as shingle and compacted silts and sands. According to the regime formula of Lacey, in which hydraulic radius is designated by R and slope by S, for regime conditions RS^2 is dependent wholly on the particle size. Accordingly, on the assumption that the existing channel is reasonably stable for dominant flow, and that the pilot cut will be in similar material, the pilot cut should be excavated to a section so that the expression RS^2 will be at least as high for the pilot cut as for the river section. As slopes of cut and of river are inversely proportional to their lengths, the workable relationship is obtained that a pilot cut will be self-scouring provided the expression R/L^2 is greater for pilot cut than for the river section, L being the length of the cut or river bend involved. This ignores the losses by shock at curves, the reduction of the discharge down the river course by the amount of the flow in the cut, and the fact that material in the cut is usually more firmly compacted. In practice, therefore, pilot cuts are designed to a rather deeper section such that R/L^2 is greater for the cut than for the original course. Incidentally, this latter expression can be obtained directly from Du Buoy's equation which states that tractive force of a river is proportional to RS. In actual practice, it is difficult to obtain the required value for scour of pilot cut unless the length of river course is $1\frac{3}{4}$ to 2 or more times the length of the cut.

The minimum hydraulic radius for scour is determined by the above expression for all practical purposes corresponds to the required depth of cut below flood level. Pilot cuts should be made as deep as possible, but in practice their depth is usually limited by the depth to which the mechanical plant available can work in water. The width is, in practice, relatively unimportant for a self-scouring cut and is usually determined by requirements of plant. For example, in shingle where bulldozers are used, the width has usually to be at least 20 ft at the bottom to enable material to be dozed out on either side — working the plant backward and forward at right angles to the line of cut. Side slopes should be designed according to the bed material. Trimming of the cut is unnecessary.

For dragline work, it is possible to go considerably deeper without trouble with water, and the section should, if possible, be made as deep as the river course and with bottom width twice the depth. This extra depth is valuable where it can be obtained, as it sometimes enables hard seams, timber, etc., to be cut through to expose looser seams beneath.

Development of cuts is usually slow at the start as the value of R is small and nearly compensates for the greater value of S. As the cut enlarges under the action of flow, its development accelerates up to a point where the cut has been enlarged to about three-fourths of the desired section. All cuts carried out in this way, therefore, require further attention some 12 months after scour has commenced, and full provision for this follow-up work should always be made, as the final development of a cut is often sluggish because the value of S falls with increase of R.

Chapter V
SUMMARY AND RECOMMENDATIONS

RIVER TRAINING

(1) No extensive river training works have been carried out in this region, except on short stretches of rivers in India for directing the flow through bridge openings and over weirs, and on a few navigable rivers in different countries connecting river ports to the sea, although the idea of training a river by confining its course within a narrow channel was suggested for the Yellow River by Pan Chishun as early as the end of the 16th century. It is well known that flood control, one of the most vital problems in this region, will surely be benefitted by river training. The future industrial development of this region is sure to expand the volume of traffic, and consequently inland navigation, which is one of the cheapest means of transportation for certain commodities, will certainly grow. The demand for river improvement will become an urgent problem. Furthermore, multiple-purpose development of water resources, which has just taken root in this region, will surely multiply and include various aspects of river regulation for the fullest use of water.

(2) River training aims essentially at the improvement of the condition of the river bed as well as the condition of flow. It is important to distinguish between bed load and suspended load, and to determine which of the two is the governing factor for river bed formation. Fargue's laws and the tractive force theory have already thrown some light on the subject but their use on account of the highly varied condition of rivers gives only qualitative rather than quantitative results. The planning and design of river training works have still to rely upon the judgement of experienced engineers. Model tests have proved to be the most helpful instrument, and systematic experimentation on the behaviour of rivers by means of models should be able to clear the way for perfecting the science of river training.

(3) For heavy sediment laden flow, both theoretical investigations and practical experience are still at a very early stage to enable the formulation of any principles to be followed in river training. More experience and study of the natural rivers in this region is urgently needed to clarify the problem. The proposed schemes for the regulation of the Yungting and the Yellow River, which comprise the contraction and stabilization of entire river courses on alluvial plains, when carired out, will throw some light to the feasibility of controlling aggrading rivers. Before any large-scale work is contemplated on these rivers, it is advisable first to test the principles on smaller streams.

(4) Bell's guide bank system has been developed in India, with success, for narrowing wide rivers at bridge openings and at diversion weirs. It appears, however, that there is still room for improvement, especially in case of rivers in flood plains where stone is scarce and expensive, or when new material or methods of construction become available. The theory of guide bund system appears sound and its application may be recommended for similar wide and wild rivers often met with in this region.

BANK PROTECTION

(1) Bank protection methods were developed in this region much earlier than river training, as the earliest record of the closure of an embankment breach was in the year 111 B.C. and the first use of woven bamboo sausages filled with stone was recorded in the year 3 B.C. It is, therefore, no wonder that a great variety of bank protection methods have been evolved in this region. A combination of local needs and conditions may result in so many different alternatives that no numerical comparison of the cost could be made to show their relative merits. In general, if a deep scour has already developed, which usually attracts the current towards it, it is preferable to divert or deflect the attacking current by the use of repelling groynes or other structures somewhere upstream rather than directly to protect the place of scour. If the river is of the aggrading type, which has usually a wide and shallow channel and carries relatively

large amount of sediment load, it is preferable to use non-continuous protection such as solid or permeable groynes. The use of continuous revetment is preferable for the meandering type of rivers where the channel is well defined, deep and winding. For both the aggrading and meandering type of rivers, when river banks are subject to erosion, it is highly desirable that protective measures be taken as early as possible rather than to tackle the problem when erosion has become serious.

(2) Regarding the suitability of material for bank protection works, keeping both the availability and cost of material in view, it appears that stone and brushwood are the most practical ones to be recommended for use in this region. Concrete and asphalt, even if available, are generally too expensive to be considered for use in this region. If boulders or cobbles are available at site, the use of wire mesh as a casing in the form of wire mesh cylinders, mattress or sausages is a practical proposition Recent experiments on the use of bricks burnt at site, or cement stabilized soil blocks, are encouraging.

(3) In the design of bank protection works, reliance is placed on experience and judgement rather than on definite laws and formulae. Analytical investigations such as the tractive force theory of Du Buoy, critical tractive force theory for particles on an inclined plane of Fan, experiments on rate of scour by a vertical jet of water of Rouse, and formulae for scour along banks and groynes of Inglis and Joglekar, have undoubtedly thrown some light on the subject, but there is still much scope for their perfection. Systematic experimentation, both on models and natural rivers, is deemed necessary.

RECOMMENDATIONS ON JOINT STUDIES

(1) In order to further our knowledge on this important problem of bank protection and river training, for the common benefit of countries of the region, joint studies by all technical organizations of the region are deemed highly desirable. Joint study on this subject may begin with the systematic recording of the behaviour of such works as well as all the hydraulic elements involved. A questionnaire has been prepared for this purpose with a view to standardizing such records so that comparable studies could be made on the same basis. Such a study, including that of the works which have already failed, will certainly disclose some relationship between the hydraulic and structural elements.

(2) Whenever a large work is to be carried out, it is strongly recommended that experimental sections may be set up with the aim of comparing various methods under the same natural conditions as was done in Hungary early in this century and in the U river in Taiwan recently. The results obtained in the course of a few years, together with the hydrological records, will certainly give valuable information, not only regarding construction methods and materials, but also regarding the fundamental principles involved in such works.

(3) On account of the changing nature of current in rivers and streams, it is strongly recommended that adequate, wise, careful and continuous maintenance be provided for any kind of river works, in particular for bank protection. Improper protection as a result of lack of maintenance is worse than no protection.

(4) Systematic experimentation in the laboratories, observations and studies on natural rivers regarding the basic subjects such as meandering, braiding, sediment transportation, scour and deposition, bank erosion and other allied items are very essential for elaborating the present day knowledge on the subject. In connexion with laboratory studies it is suggested that research programmes could be co-ordinated and work divided among the various technical organizations of the region with a view to avoiding overlapping of work and economizing the resources available.

APPENDIX

Questionnaire on River Training and Bank Protection

I. WHAT ARE THE EXISTING NATURAL CONDITIONS?

1. (*a*) What kind of materials are the banks and the bed made of?

   (*b*) What are the variations in the bank materials locally, and on the whole length of the alluvial river?

2. *Hydraulic factors*

   (*a*) Discharges (maximum, minimum and average)

   (*b*) Width (bankful stage, mean water (MW), mean low water (MLW)

   (*c*) Average velocity during HW, MW and LW

   (*d*) Slope (MHW, MW, MLW)

   (*e*) Average depth along the eroded bank, below mean low water

   (*f*) Average height of bank above mean low watel

   (*g*) Differen in water level between MHW, bankful stage and MLW

   (h) Sediment Rate during MHW, MW and MLW

3. *Meandering*

   (*a*) Length and radius of bends where erosion is severe and where erosion is moderate; nature of soil at bends

   (*b*) Degree of sinuosity or percentage of meandering, i.e. ratio of the talweg length to the airline distance

   (*c*) Is there a general limiting width to the lateral development of bends (according to information given by old maps or by aerial photographs, etc.)?

   (*d*) Do the summit of bends migrate down or up the valley?
   Rate of migration

   (*e*) Period of erosion; prevailing slope, velocity, depth of rivers at bends during erosion

4. *Bank erosion*

   (*a*) Degree of erosion, i.e., ratio of the length of eroded banks (on both sides) to the length of the river

   (*b*) Soil of bank

   (*c*) Rate of erosion of different bends from the very beginning of erosion
   Maximum (in one year)

   (*d*) Variations of this rate and of the point of attach with changes in stage (high or low water, rising or subsiding levels), and slope of the river

   (*e*) Where are the eroded materials deposited? Immediately in the pools downstream from the point of erosion, or along the edges of the bars, or along the top of convex bars, or further downstream?

   (*f*) Period of erosion; prevailing slope, velocity and depth of river at places of erosion

5. *Natural cut-offs*
   (*a*) Number
   (*b*) Cause; do they result from local differences in the erodibility of bank materials?
   (*c*) Rate of development (change of width, depth and velocity) ....
   (*d*) Effects

6. *Available material*
   (*a*) What is the material (stone, wood, bamboo, stalks, grasses, etc.), available for bank protection?
   (*b*) How far from the eroded banks?

II. BANK PROTECTION.

1. *What is the method in use*
   (*a*) To protect directly the points of attack
   To try to guide the river upstream
   (*b*) Temporary or permanent
   (*c*) Spurs (or groynes) or revetments

2. *Spurs (or groynes)*
   When carried out
   The materials made of
   Permeable or not
   Length
   Longitudinal profiles
   Cross-section of the body
   Cross-section of the head
   Alignment in comparison to the main current
   Spacing
   Protection against scouring
   Cost of construction per unit length of protected bank
   Cost of maintenance per unit length of protected bank

3. *Revetment*
   When carried out
   The materials used
   Characteristics (height, depth, slope, etc.)
   Apron or protection against scouring
   Cost of construction per unit length of protected bank
   Cost of maintenance per unit length of protected bank

4. *Guiding the river*
   (*a*) Method (groyne, artificial island, guide banks, etc.)
   (*b*) How far from the point of attack
   (*c*) Characteristics (See II 2 and II 3)

5. *Results*
   (*a*) Incident or casualties during or after construction
   (*b*) Effect on bank erosion
   (*c*) Effect on channel
   (*d*) Effect on migration down or up the valley of the bends
   (*e*) Are the expenses (construction and maintenance) justified by results (value of protected properties including dikes, stability of navigable channel, etc.)?

III. ANY OTHER INFORMATION OR OPINION ABOUT BANK PROTECTION.

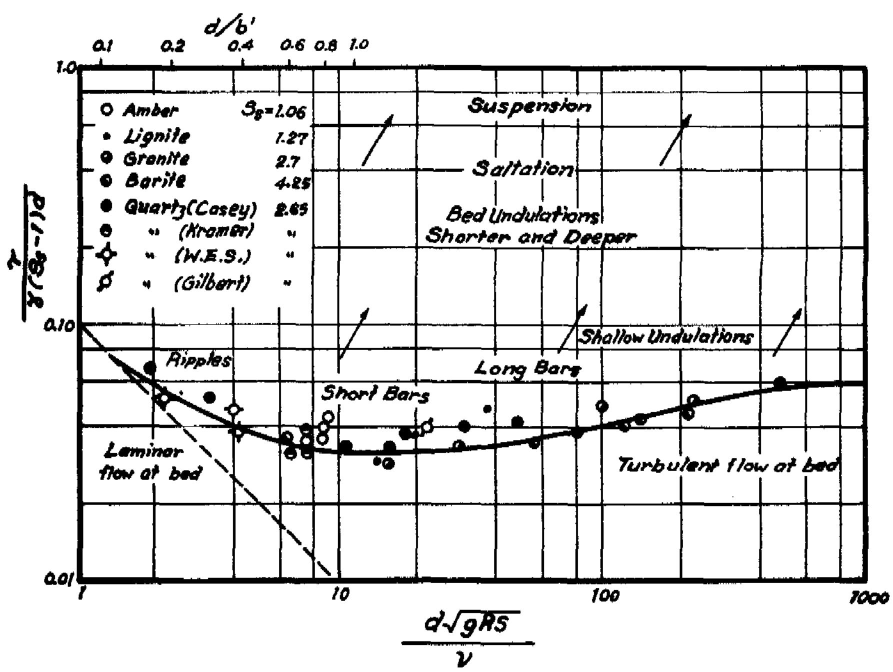
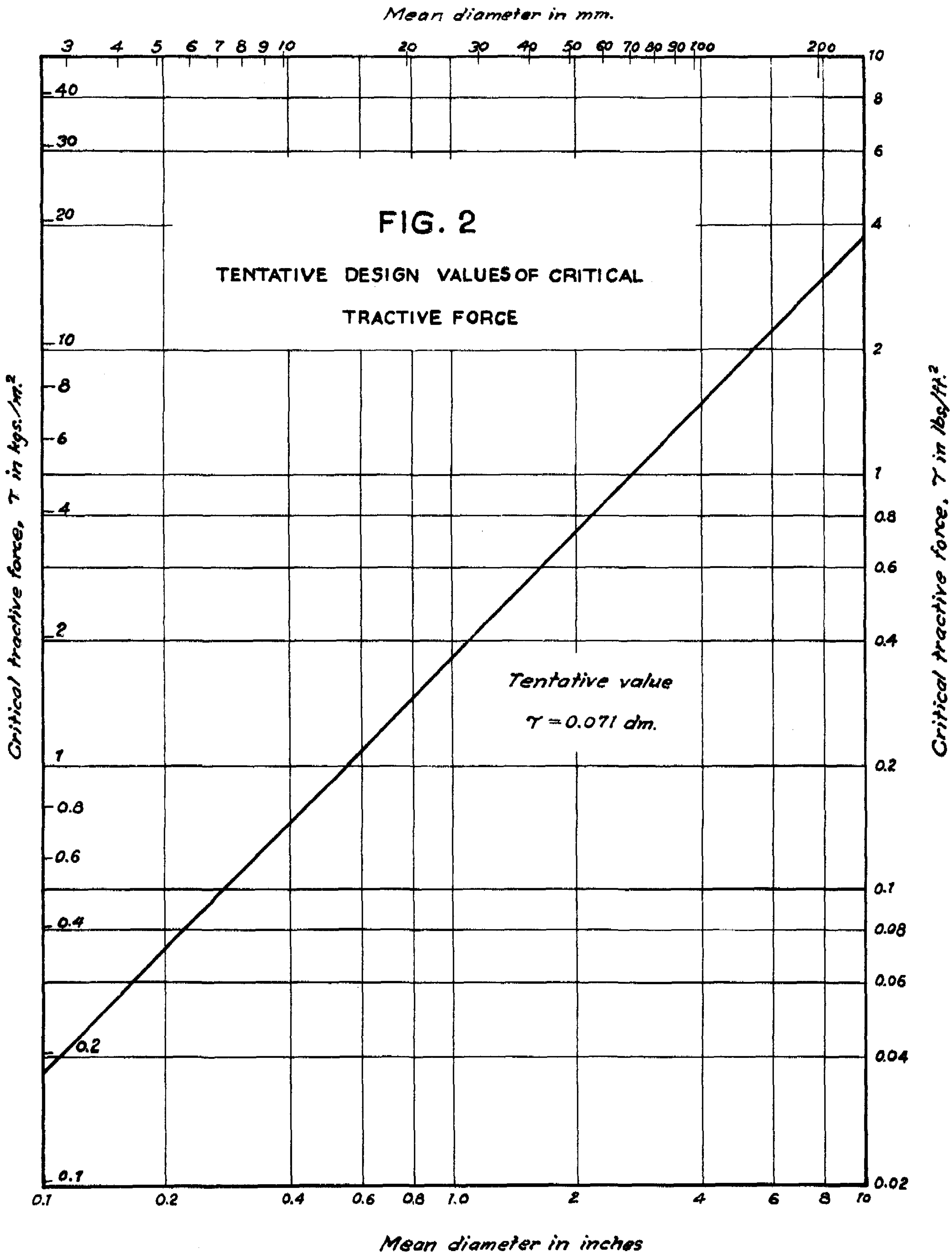


FIG. 1 RELATION BETWEEN CRITICAL TRACTIVE FORCE AND RIVER BED MATERIAL



Mean diameter in mm.
3 4 5 6 7 8 9 10 20 30 40 50 60 70 80 90 100 200
FIG. 2
TENTATIVE DESIGN VALUES OF CRITICAL TRACTIVE FORCE
Tentative value
τ = 0.071 dm.
Critical tractive force, τ in kgs./m.²
40 30 20 10 8 6 4 2 1 0.8 0.6 0.4 0.2 0.1
Critical tractive force, τ in lbs/ft.²
10 8 6 4 2 1 0.8 0.6 0.4 0.2 0.1 0.08 0.06 0.04 0.02
0.1 0.2 0.4 0.6 0.8 1.0 2 4 6 8 10
Mean diameter in inches

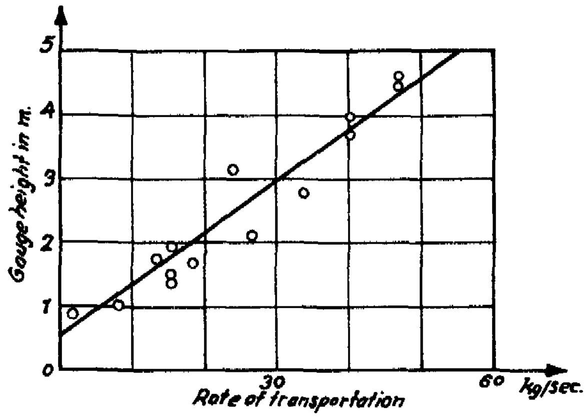


FIG. 3 BED LOAD TRANSPORTATION OF RIVER WEICHSEL AT THORN

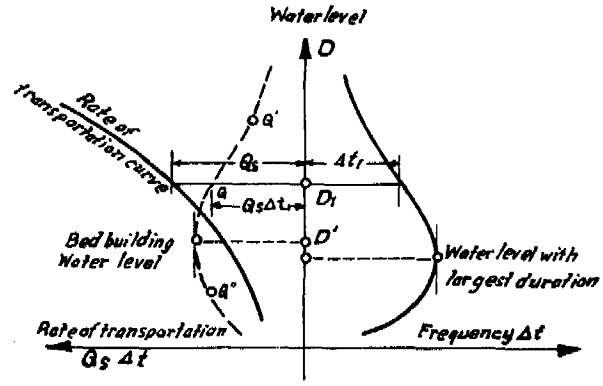


FIG. 4 DETERMINATION OF BED BUILDING STAGE OF RIVER

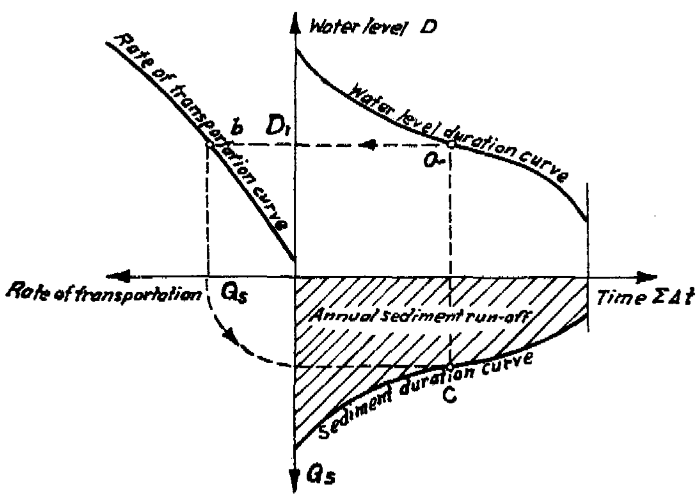


FIG. 5 DETERMINATION OF ANNUAL SEDIMENT RUN-OFF

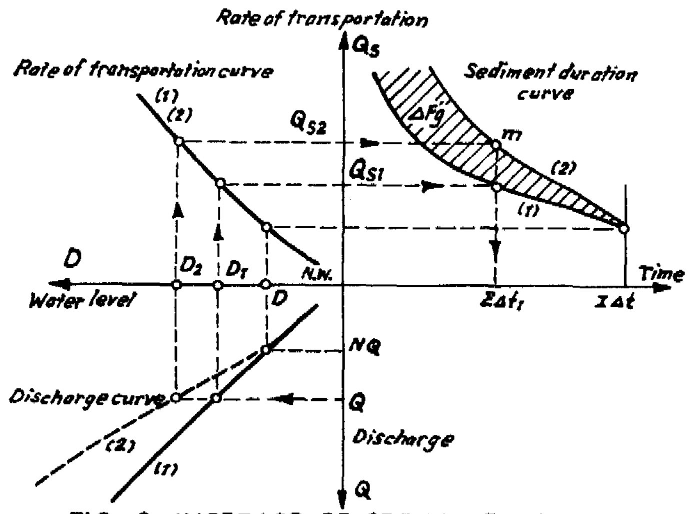


FIG. 6 INCREASE OF SEDIMENT RUN-OFF DUE TO CHANNEL CONTRACTION

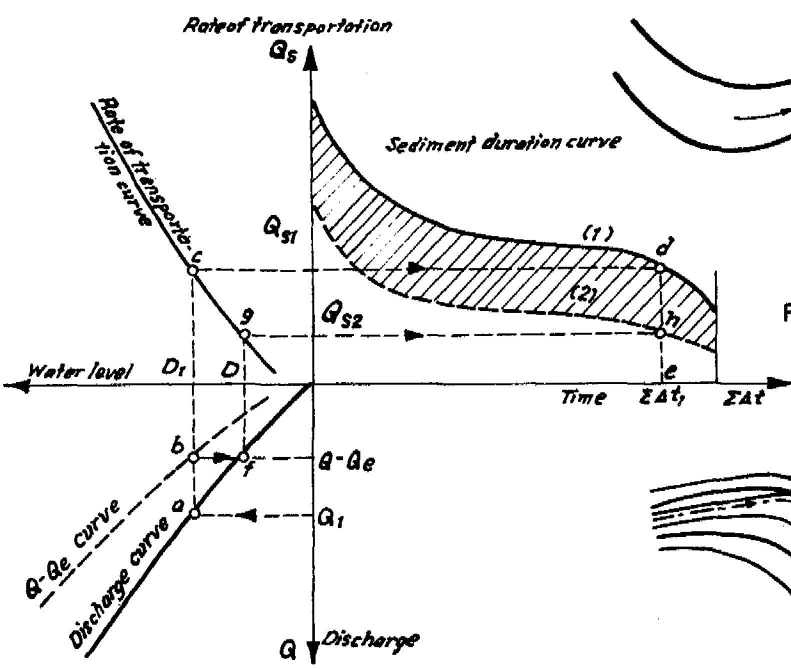


FIG. 7 DECREASE OF SEDIMENT RUN-OFF DUE TO WITHDRAWAL OF WATER FROM RIVER

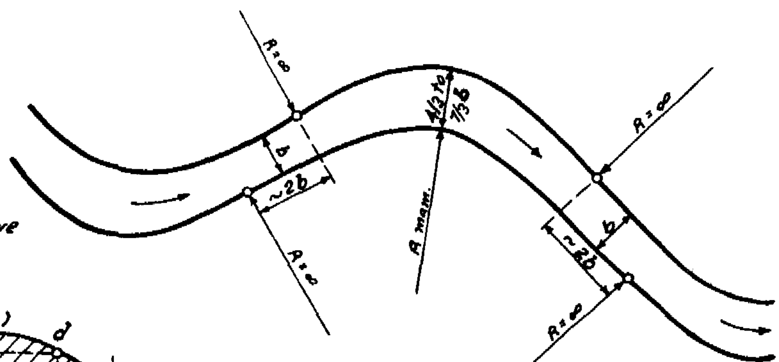


FIG. 8 ALIGNMENT OF MEAN BED CHANNEL

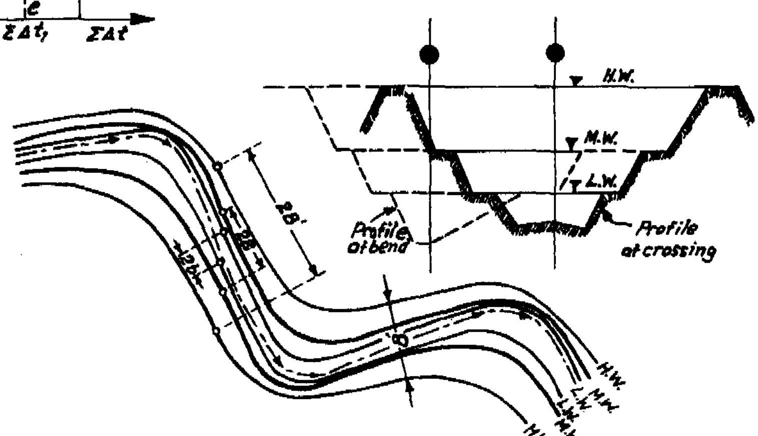


FIG. 9 IDEAL ALIGNMENT OF LOW, MEAN AND HIGH WATER CHANNEL

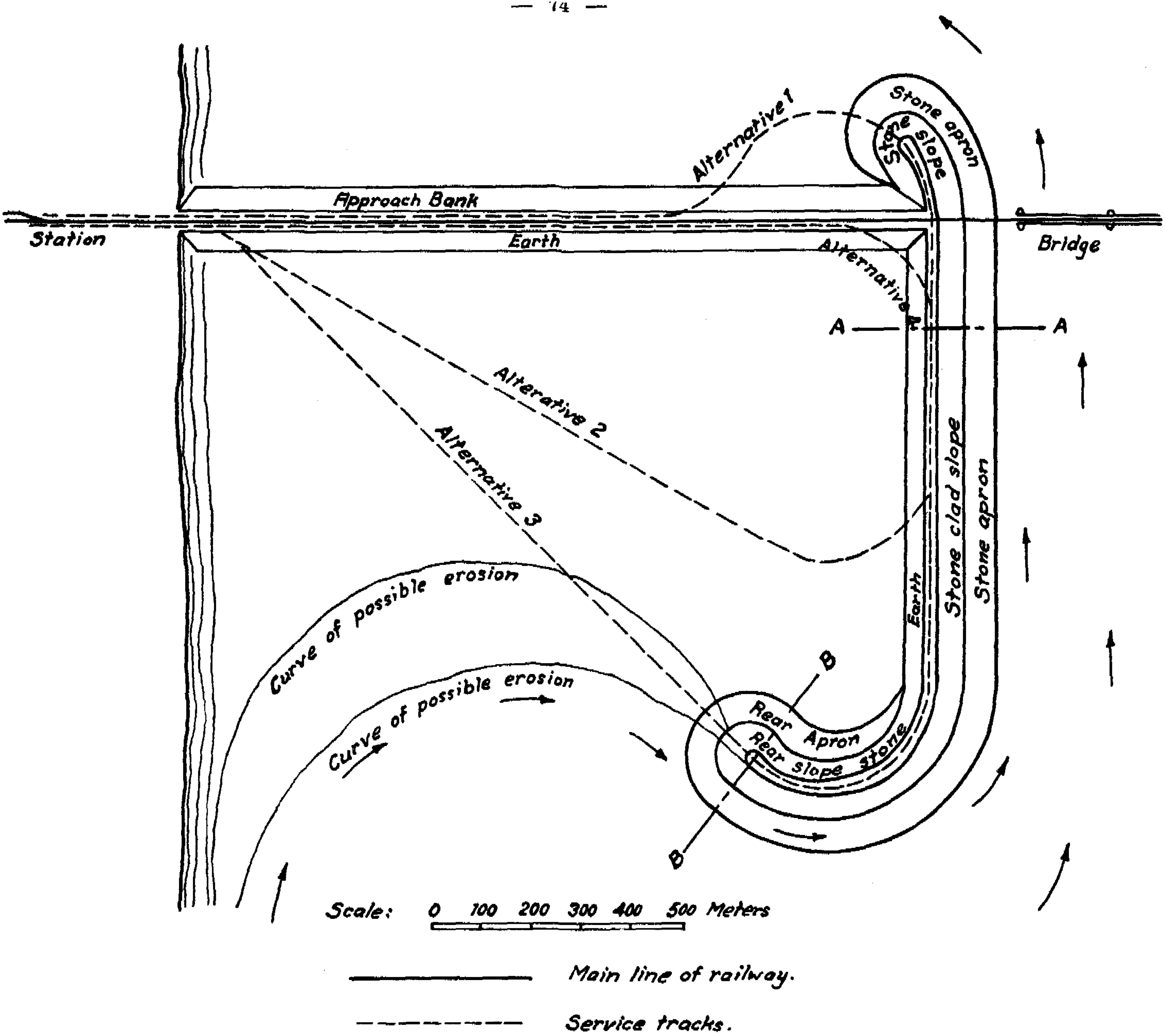


FIG. 10 ARRANGEMENT FOR THE MAINTENANCE OF GUIDE BANKS

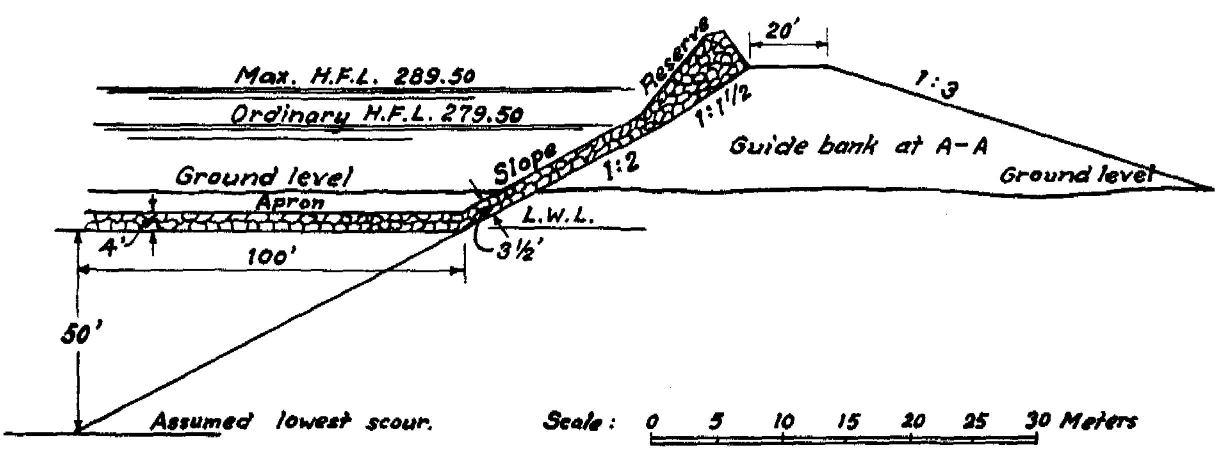


FIG. 11 CROSS-SECTION OF GUIDE BANK AT ALLAHABAD

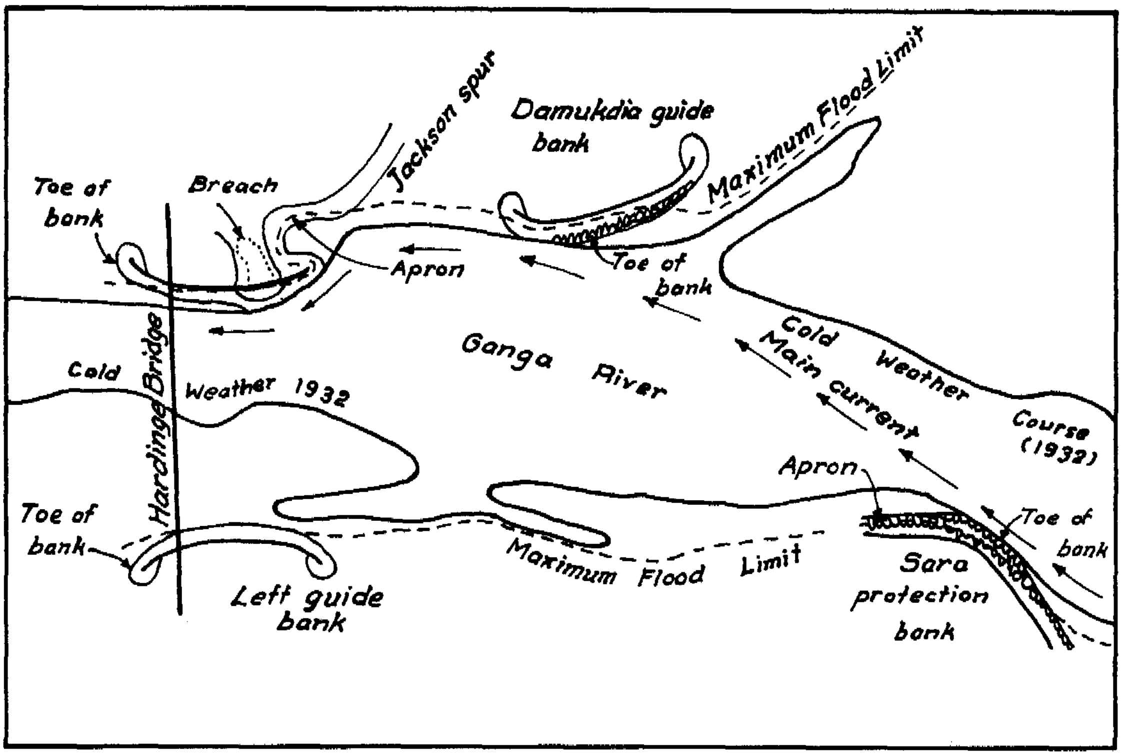


FIG. 12 HARDINGE BRIDGE ON THE GANGA RIVER

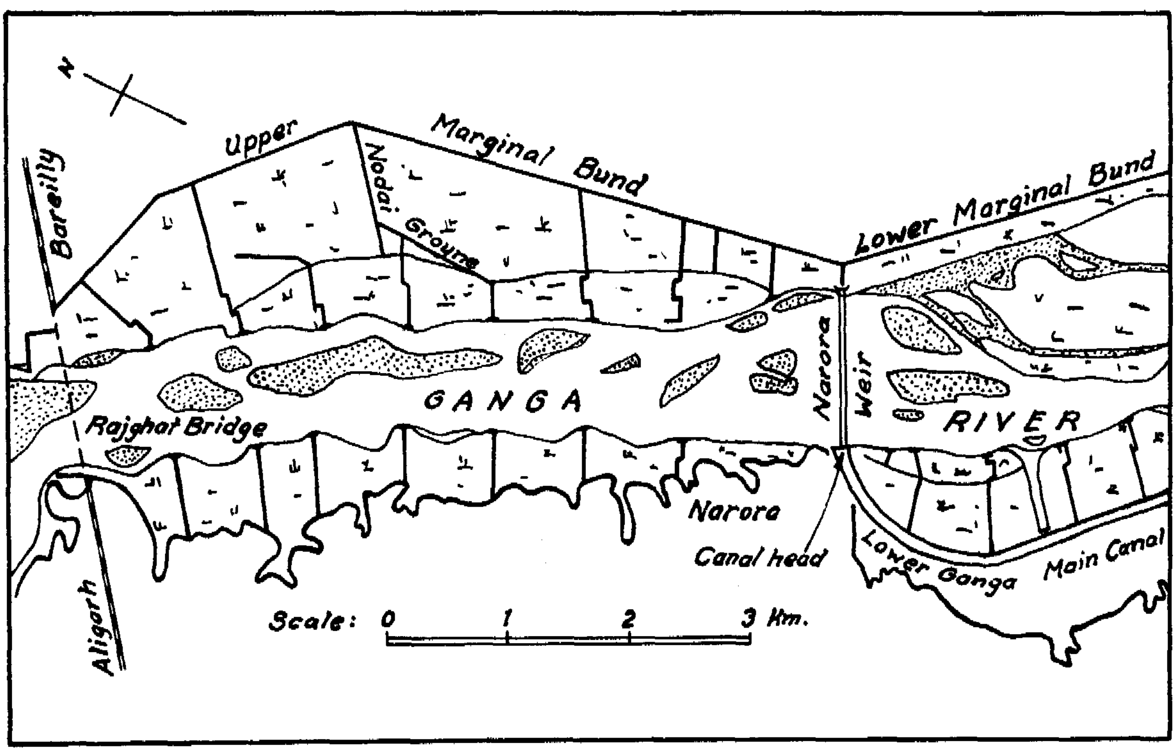


FIG. 13 DENEHY'S GROYNE AT NARORA CANAL HEADWORK

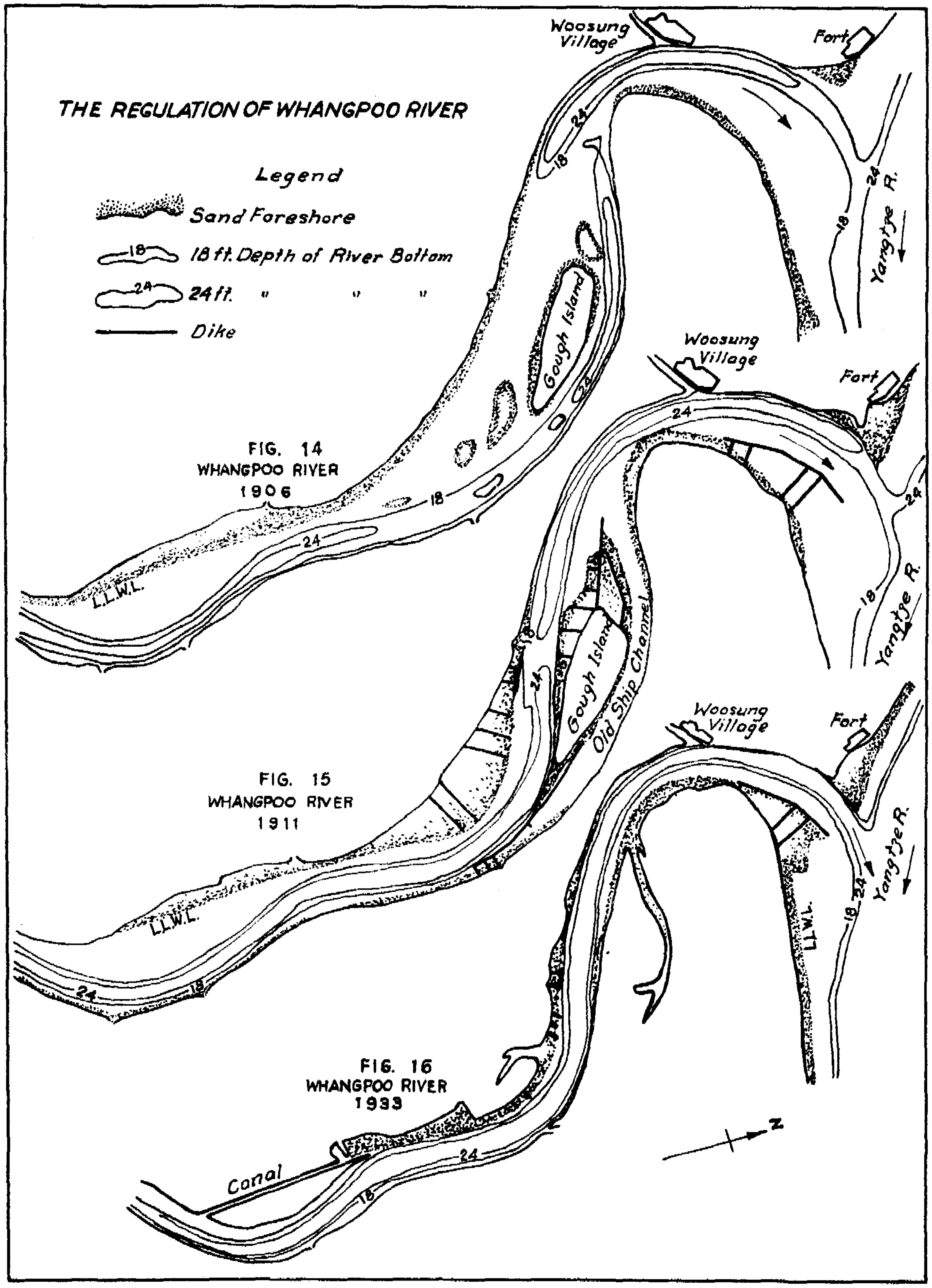
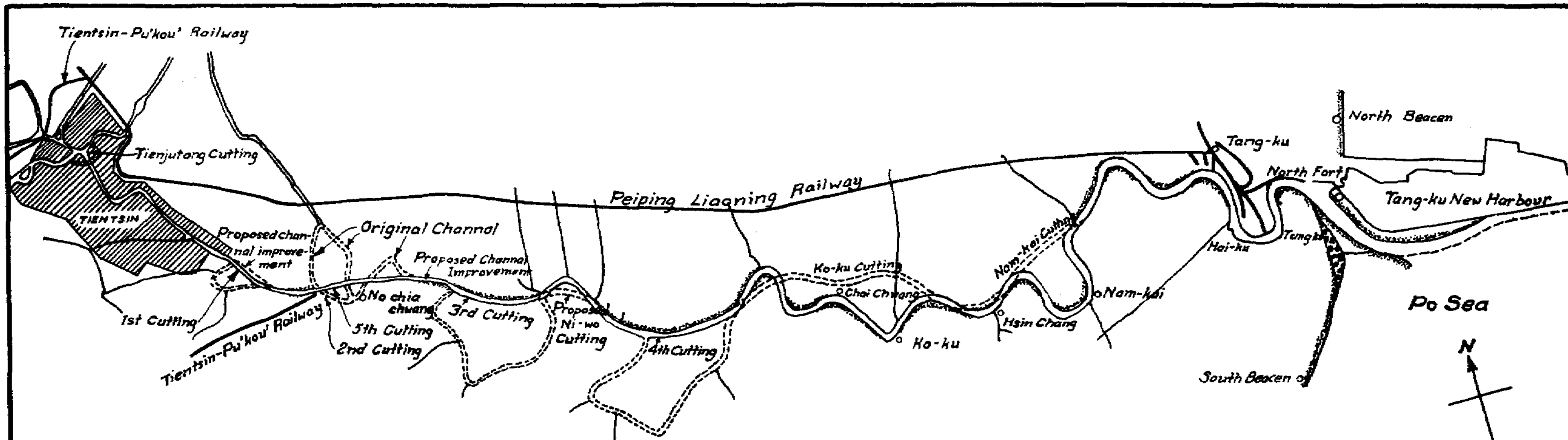


FIG. 14
WHANGPOO RIVER
1906

FIG. 15
WHANGPOO RIVER
1911

FIG. 16
WHANGPOO RIVER
1933



NOTE

Original distance by river, Taku North Fort to Tientsin Harbour Master Office = 90.44 Km = 48.8 Nautical Miles.
Present (1933) " " " " " " " " " " " = 65.19 Km. = 35.2 " "

Name of cutting		Original length	Present length
1st cutting	(1901-1902)	11,800 ft	5,000 ft.
2nd "	(1901-1902)	23,000	6,000
3rd "	(1903-1904)	35,000	11,600
4th "	(1911-1913)	43,000	12,400
Tienjutang "	(1918-1919)	6,000	800
5th "	(1921-1923)	13,000	10,000

The average Tidal Range at Tientsin during May & June (the Low Water Period of the River & Tributaries) has been as follows:-

1898 - Before conservency work	0.31 ft.
1902 - After Closing Chan Shine How, Chun Liang Cheng, Hsi Ku by Locks	1.52 "
1903 - After opening 1st & 2nd cuttings	1.88 "
1905 - After opening 3rd cutting	2.58 "
1912 - When the first 3 cutting had been opened 7 yrs.	3.58 ft.
1913 - Before opening 4th cutting	3.87 "
1914 - After opening 4th cutting	4.39 "
1919 - When the 4th cutting had been opened 6 yrs.	6.11 "
1920	6.25 "
1945	6.48

FIG. 17
CUT-OFFS OF THE HAI HO
FROM TIENTSIN CITY TO THE SEA
1 0 1 2 3 4 5 Km.

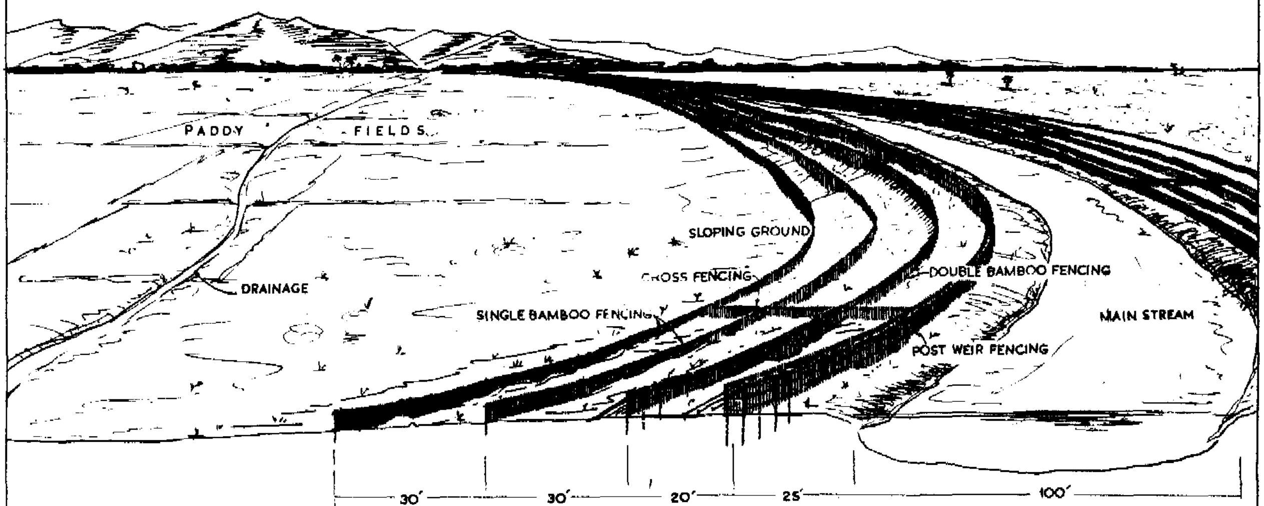


FIGURE 18 RIVER TRAINING WITHOUT EMBANKMENT IN BURMA
(AMELIORATION DU LIT SANS ENDIGUEMENT EN BIRMANIE)

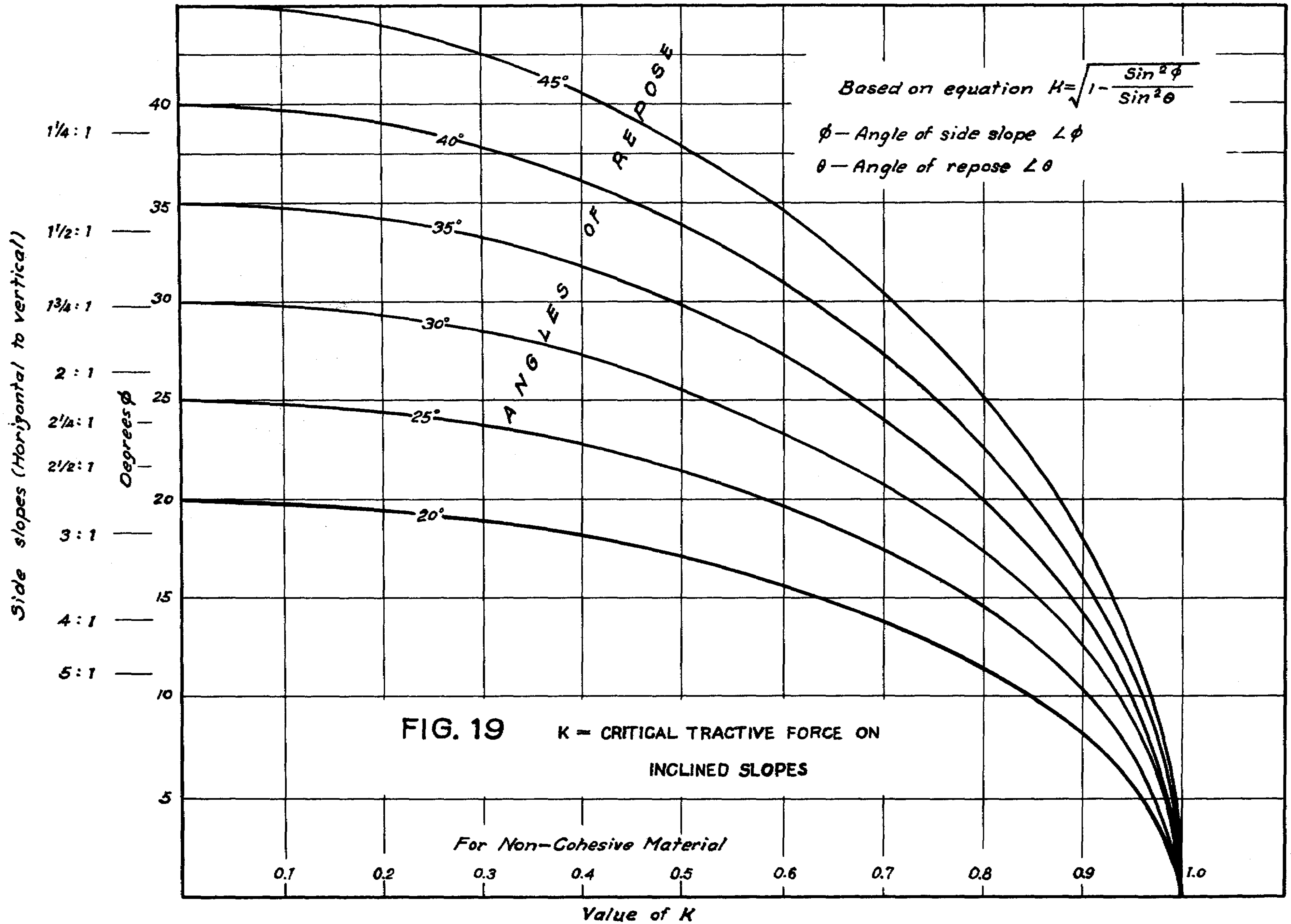


FIG. 19 K = CRITICAL TRACTIVE FORCE ON INCLINED SLOPES

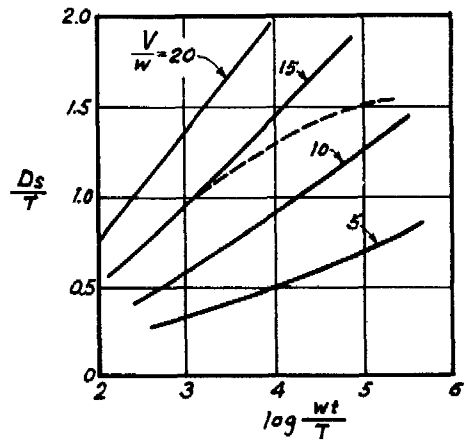


FIG. 20 DEPTH OF SCOUR AS A FUNCTION OF SIZE AND VELOCITY OF JET, TIME, AND FALL VELOCITY OF SEDIMENT

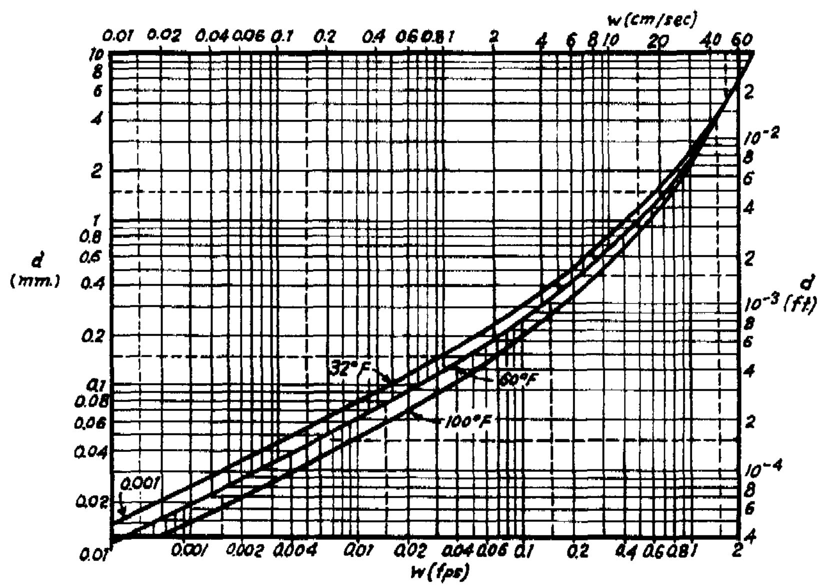


FIG. 21 CHART OF SEDIMENTATION DIAMETER VERSUS FALL VELOCITY IN WATER

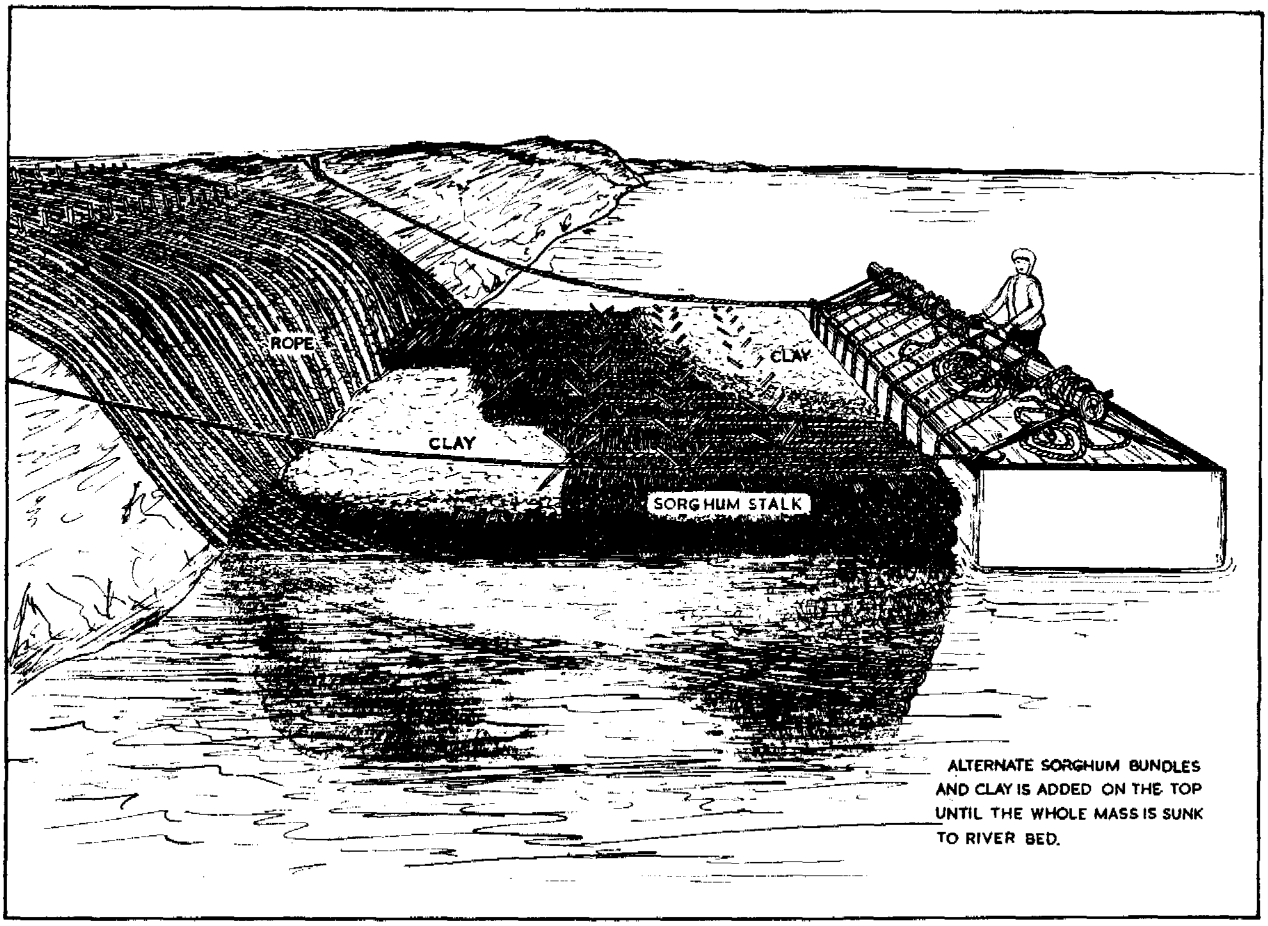


FIGURE 22 CONSTRUCTION OF SORGHUM REVETMENT IN YELLOW RIVER.

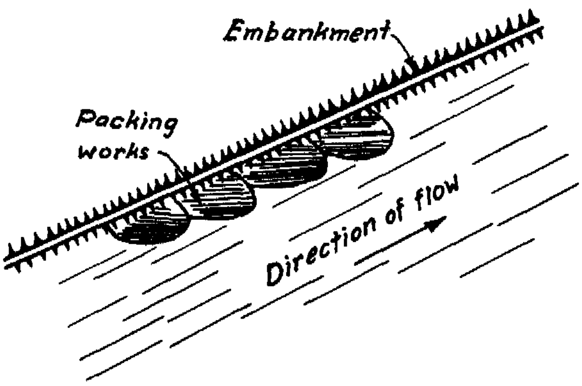


FIG. 23 THE ARRANGEMENT OF FISH SCALE PACKING

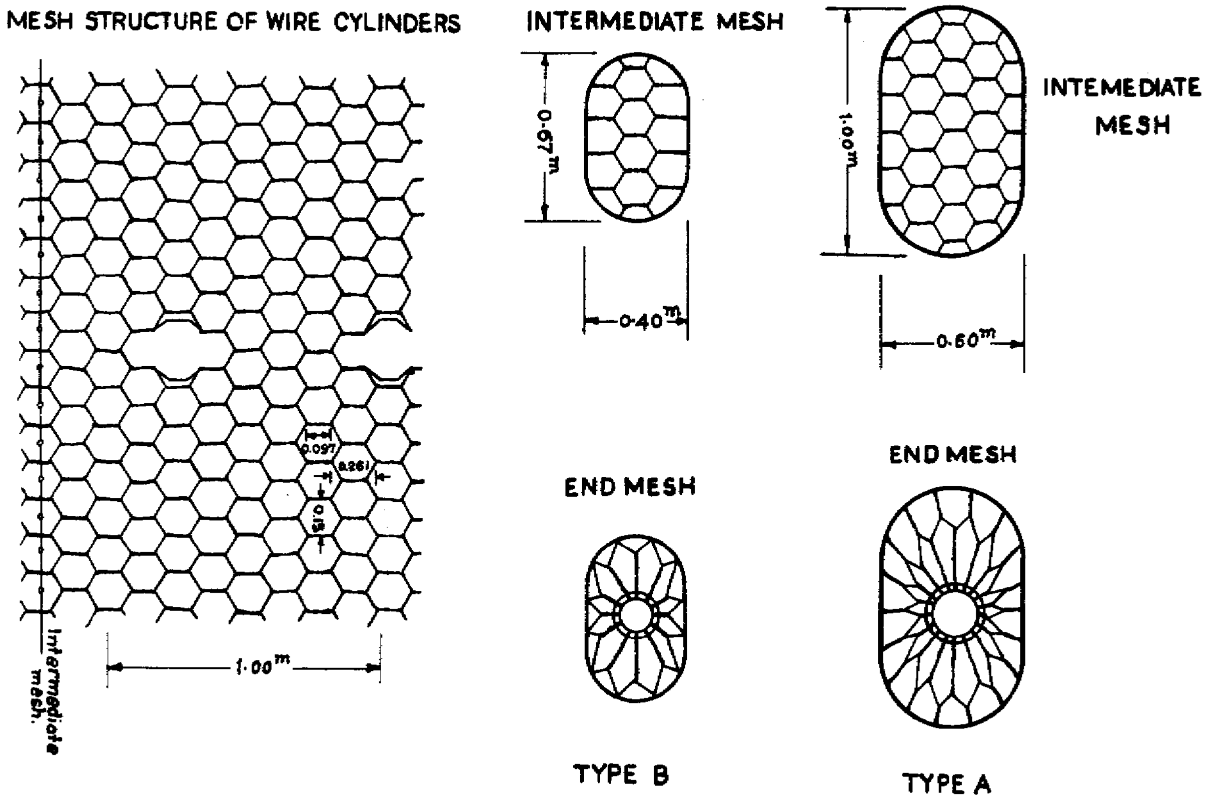
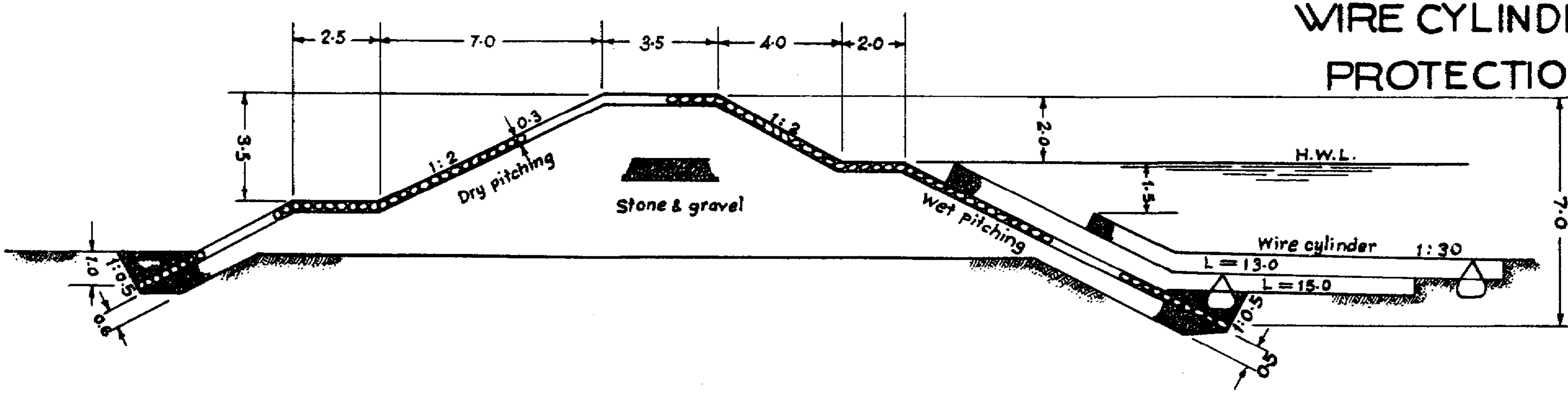


FIG. 24

WIRE MESH CYLINDER

FIG. 25

STONE LEVEE
WIRE CYLINDER
PROTECTION



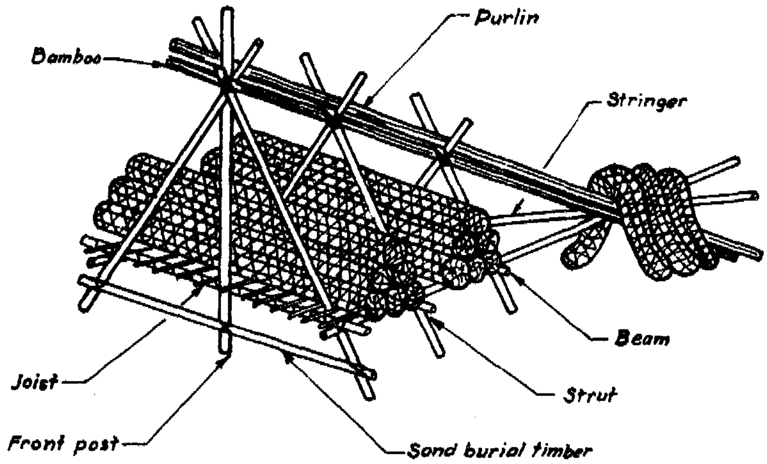


FIG. 26 SACRED COW

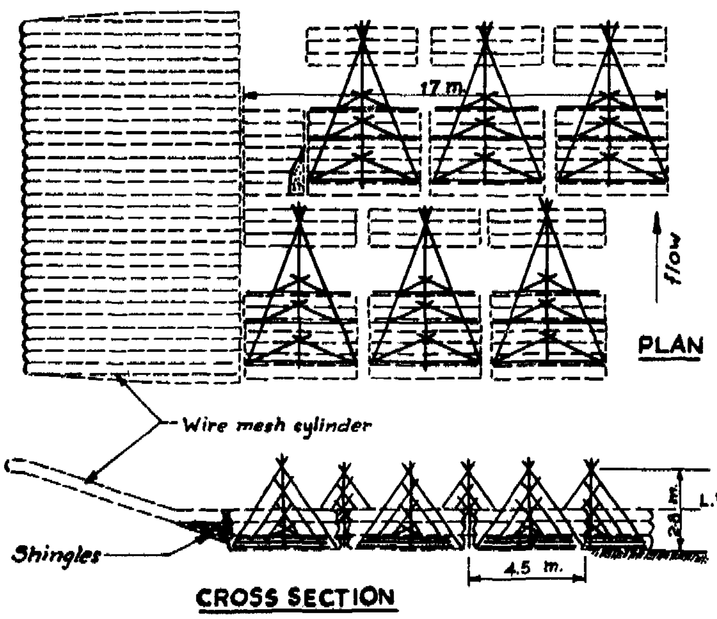


FIG. 27 ARRANGEMENT OF SACRED COW

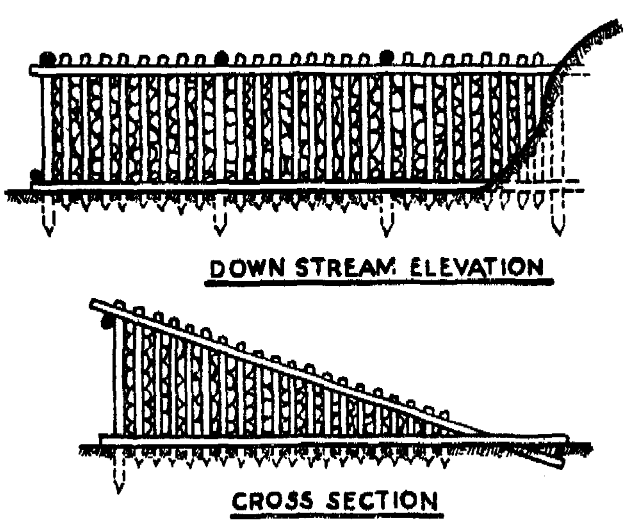


FIG. 28 WEDGE SHAPED CRIB

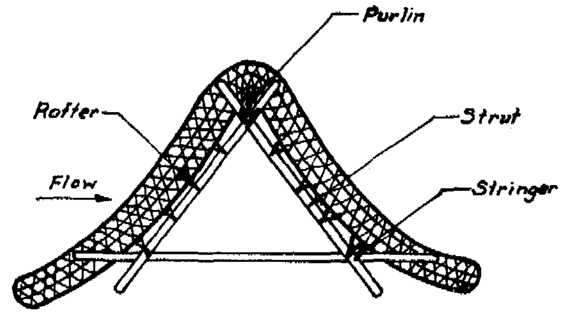


FIG. 29 STABLED COW

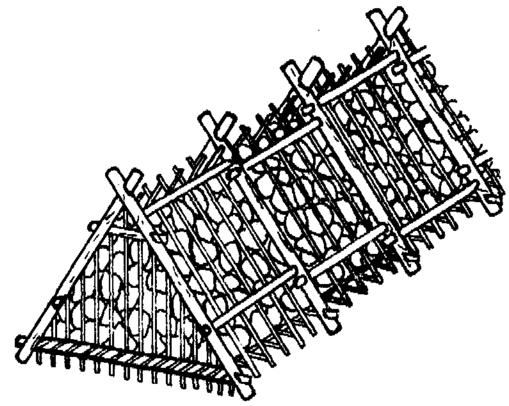


FIG. 30 TRIANGULAR PRISM CRIB

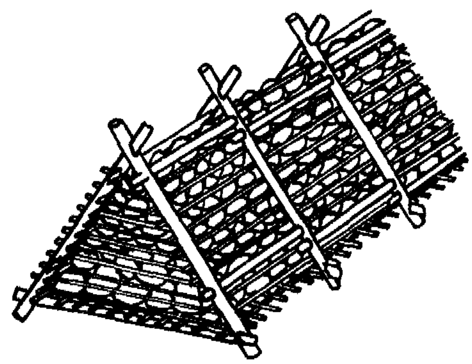


FIG. 31 TRIANGULAR PRISM CRIB

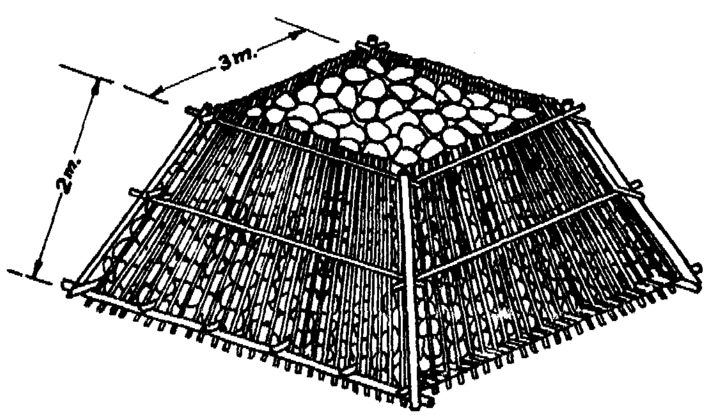


FIG. 32 FRUSTUM CRIB

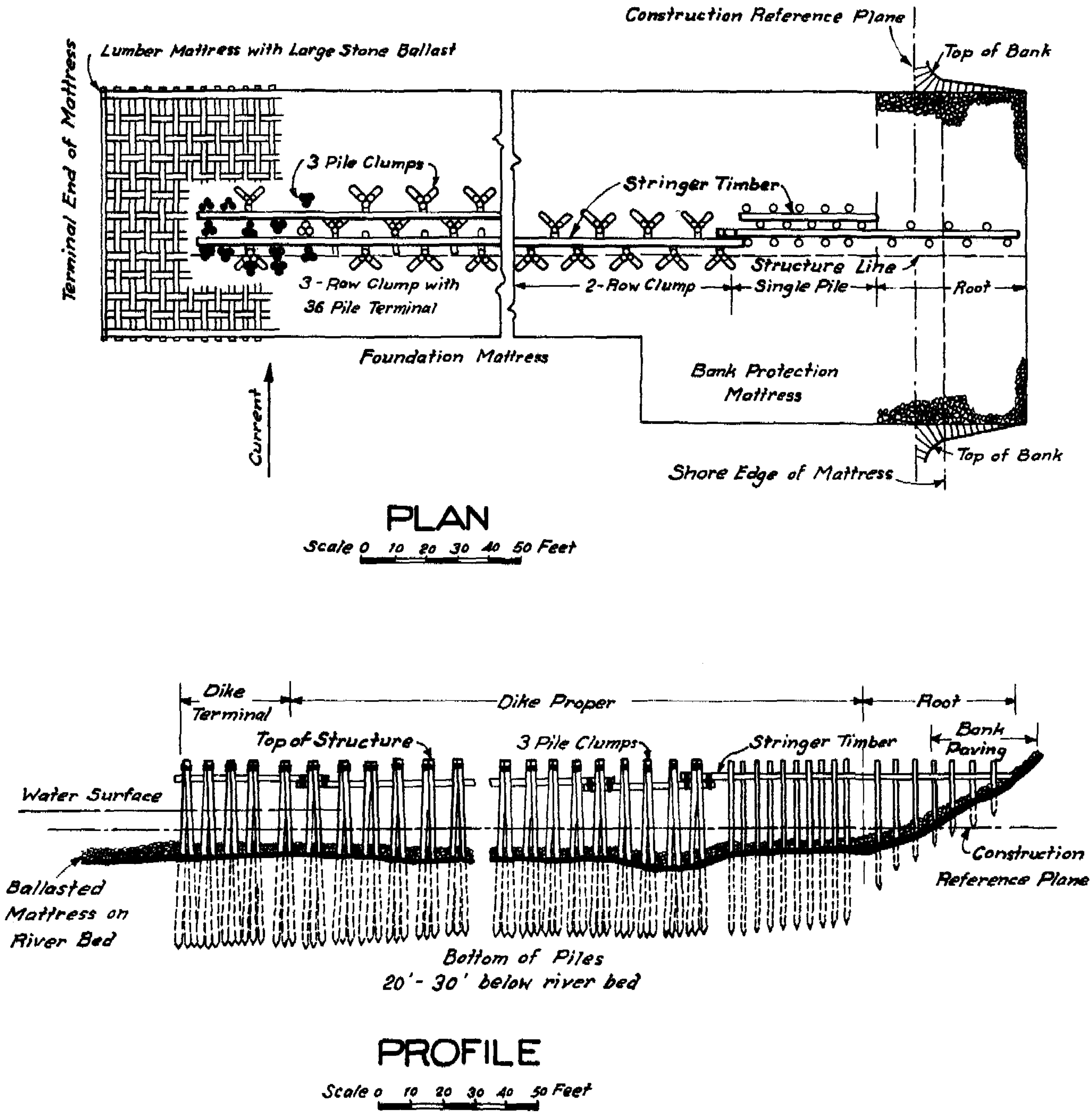


FIGURE 33 STANDARD PILE CLUMP GROYNE

BANK PROTECTION

MISSISSIPPI RIVER

SYSTEM

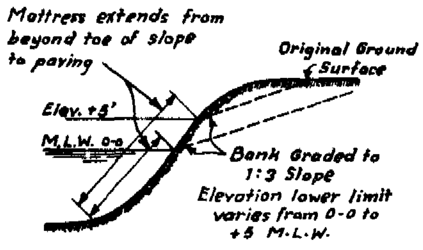


FIG. 34
TYPICAL CONSTRUCTION PROFILE

UPPER MISSISSIPPI RIVER TYPE

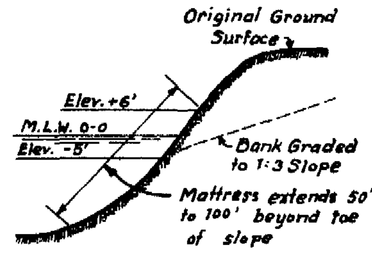


FIG. 35
TYPICAL CONSTRUCTION PROFILE

LOWER MISSISSIPPI RIVER TYPE

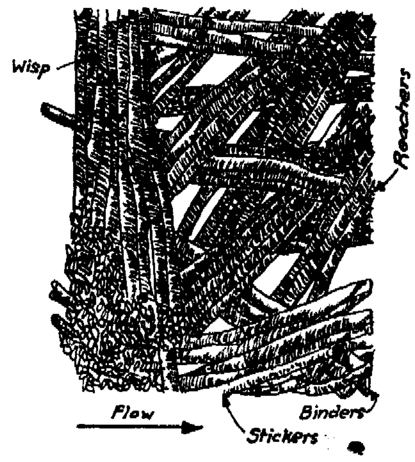


FIG. 36 WOVEN WILLOW MATTRESS

MISSOURI RIVER TYPE

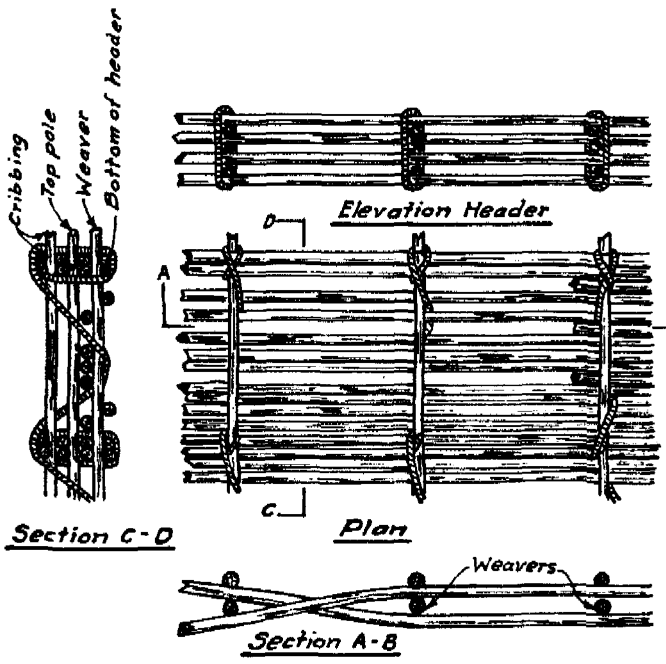


FIG. 37 WOVEN WILLOW MATTRESS

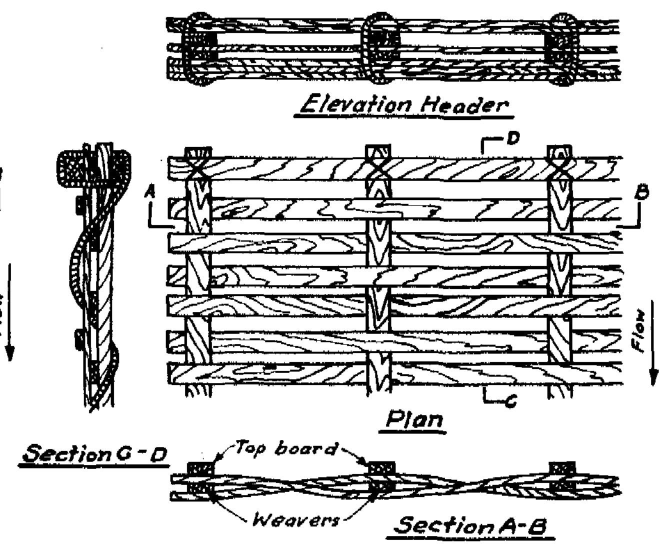


FIG. 38 WOVEN LUMBER MATTRESS

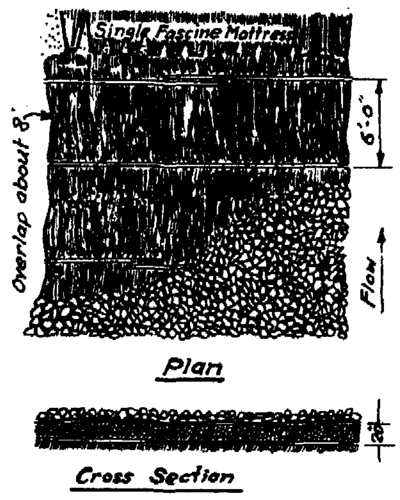


FIG. 39 FASCINE MATTRESS

UPPER MISSISSIPPI RIVER TYPE

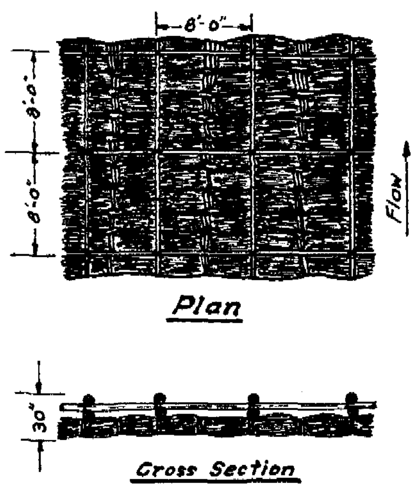


FIG. 40 FASCINE MATTRESS

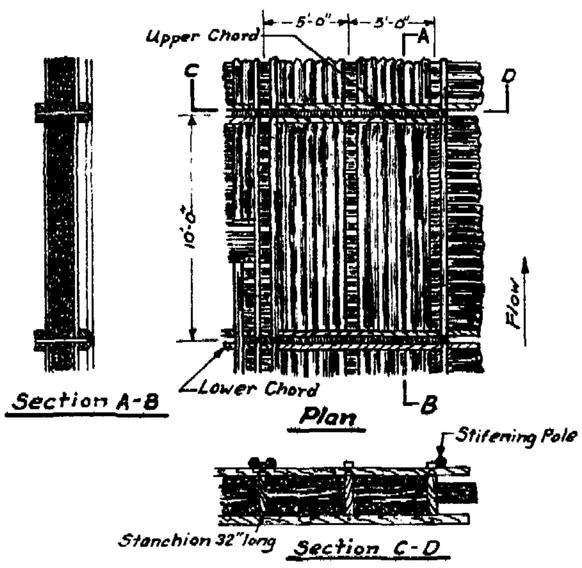


FIG. 41 WILLOW FRAMED MATTRESS

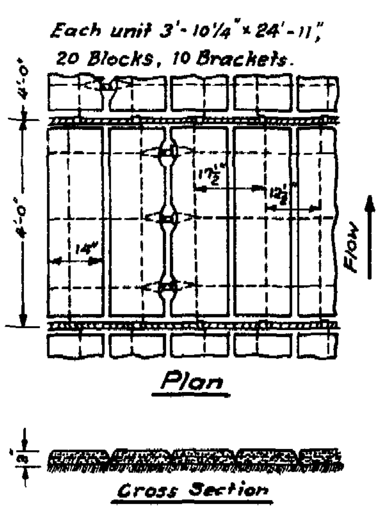


FIG. 42
ARTICULATED CONCRETE MATTRESS

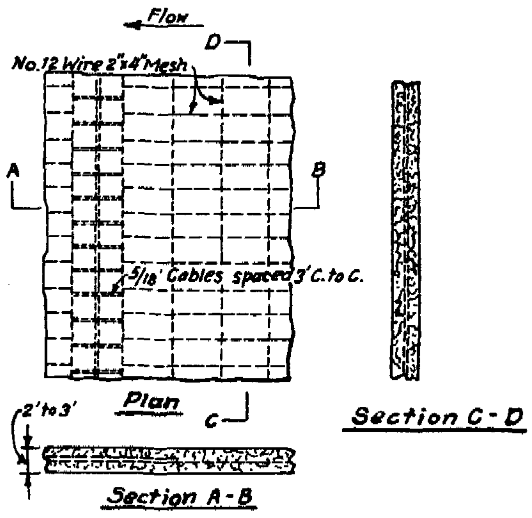


FIG. 43 ASPHALT MATTRESS

LOWER MISSISSIPPI RIVER TYPE

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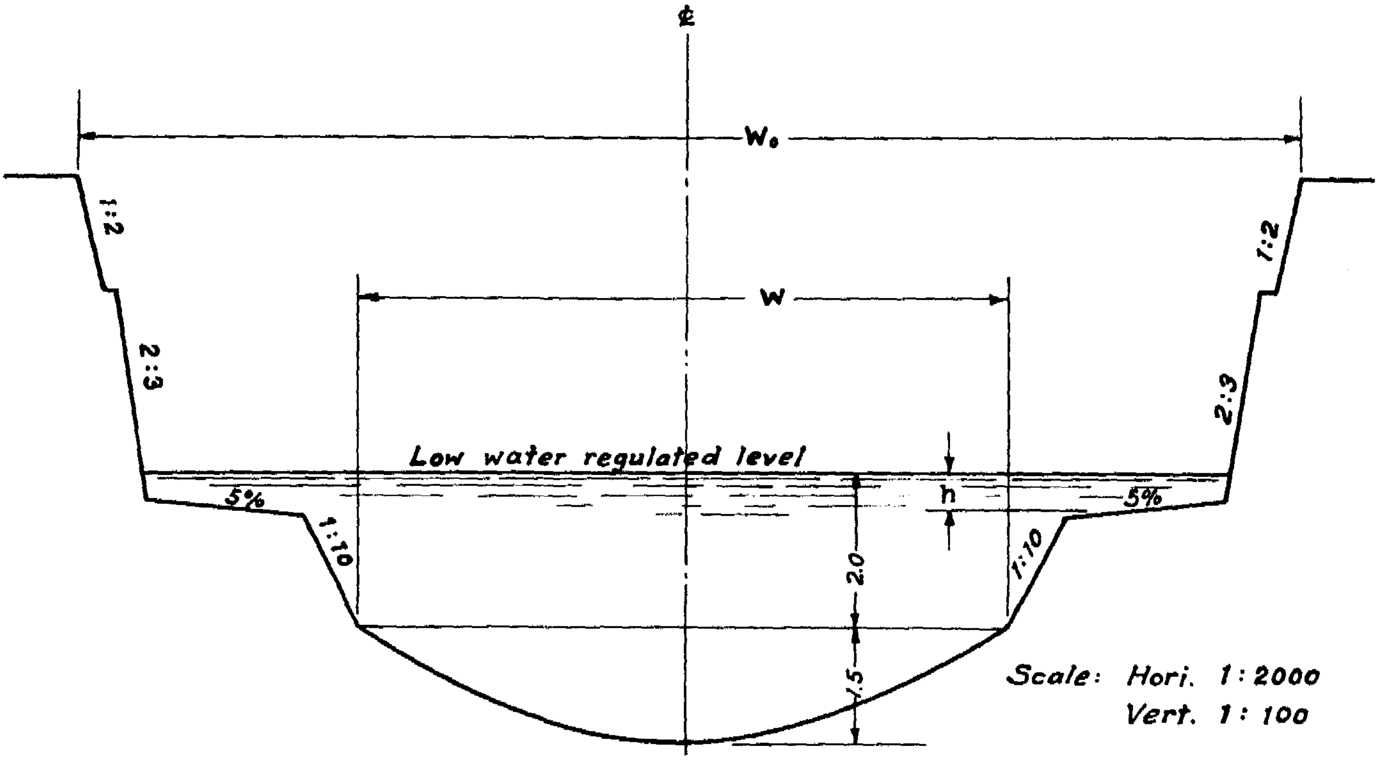
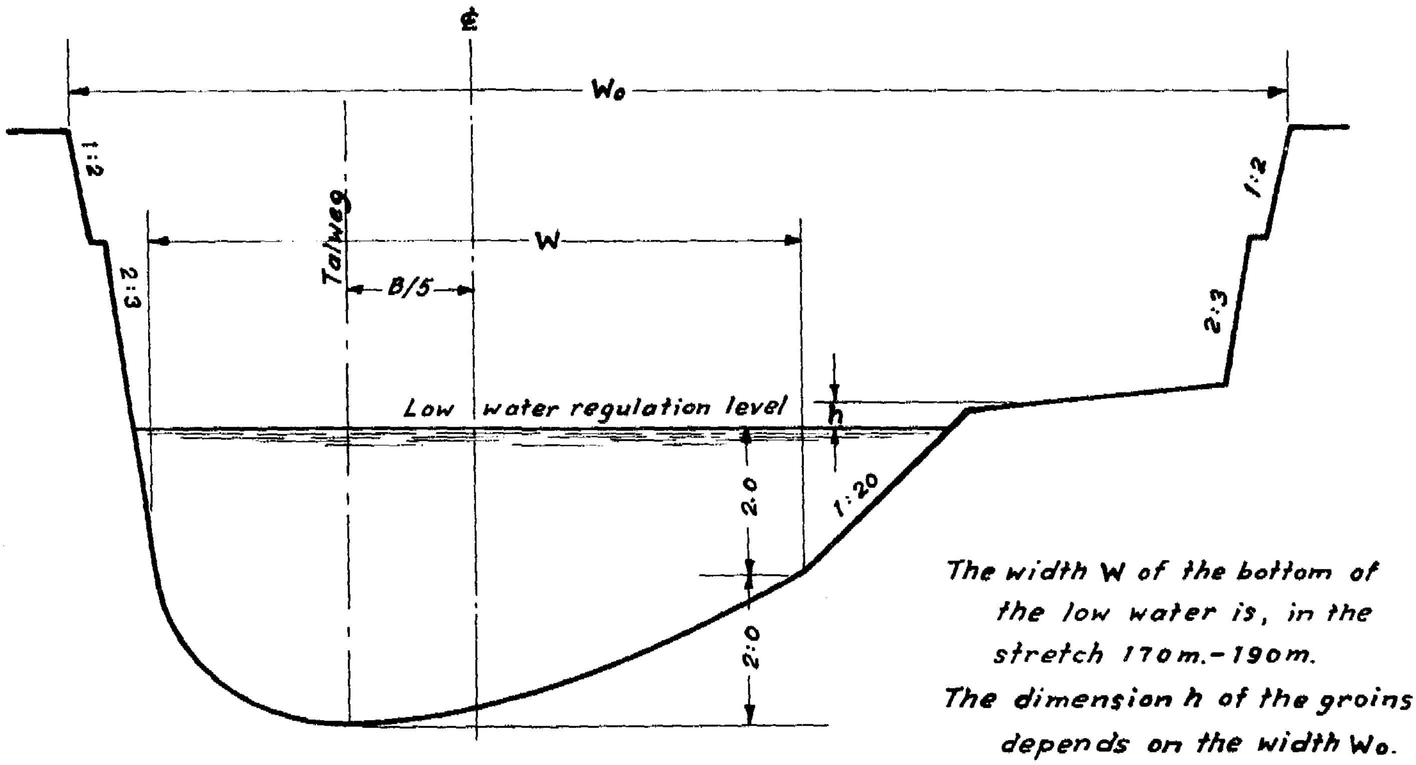


Cross-Section at the Crossing



Cross-Section near the Top of the Bends

FIG. 44 DESIGNED CROSS-SECTIONS OF RIVER DANUBE

(BETWEEN KM. 1879.9–2200.6)

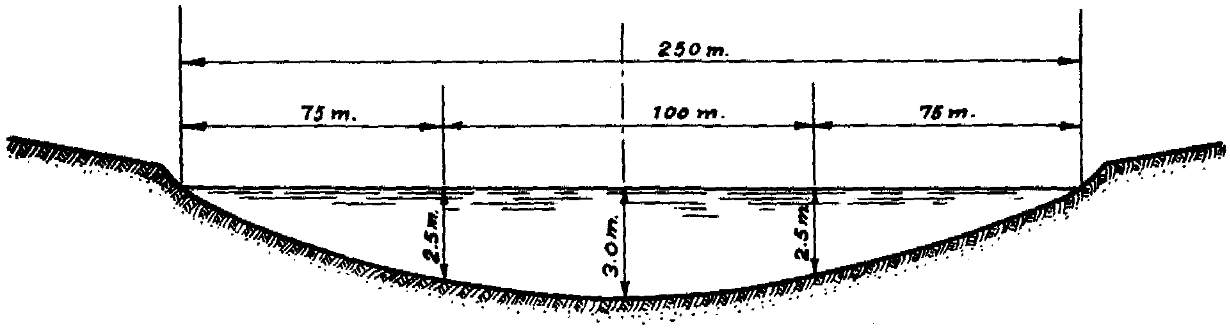


FIG. 45 PROPOSED NORMAL SECTION OF REGULATED CHANNEL FOR THE RIVER PO

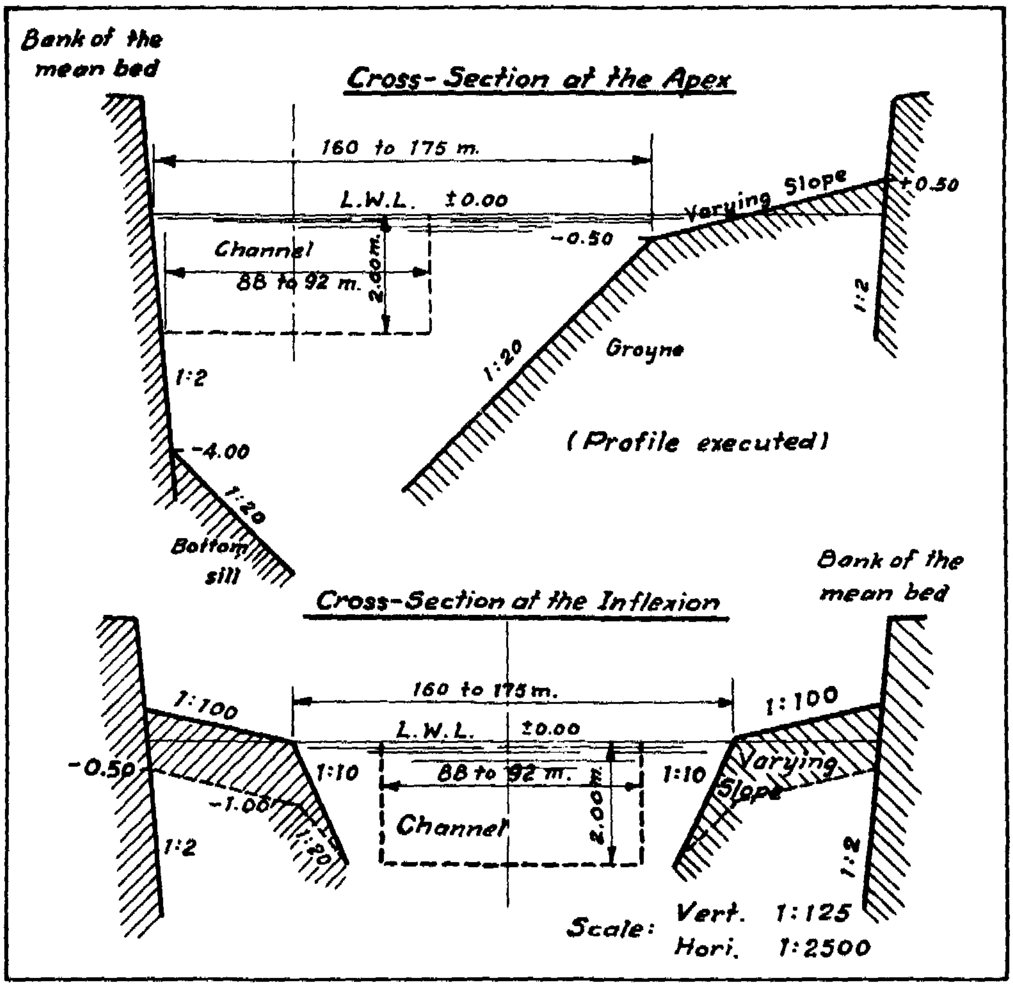


FIG. 46 REGULATION OF THE RHINE

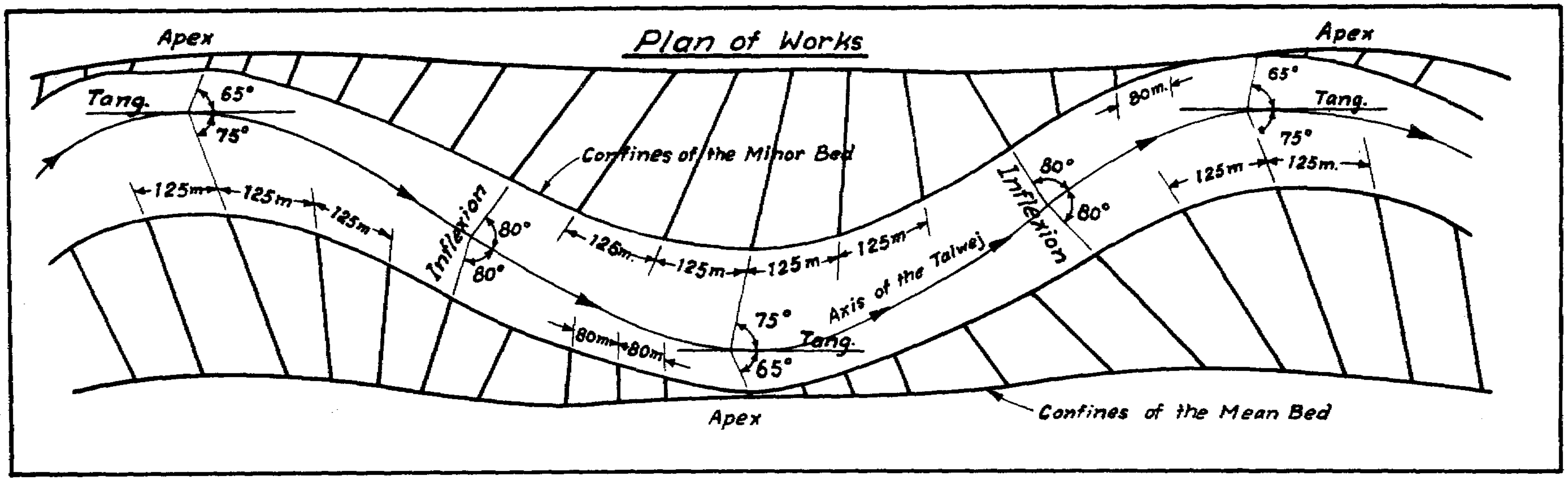


FIG. 47 REGULATION OF THE LOIRE BETWEEN ANGERS AND NANTES

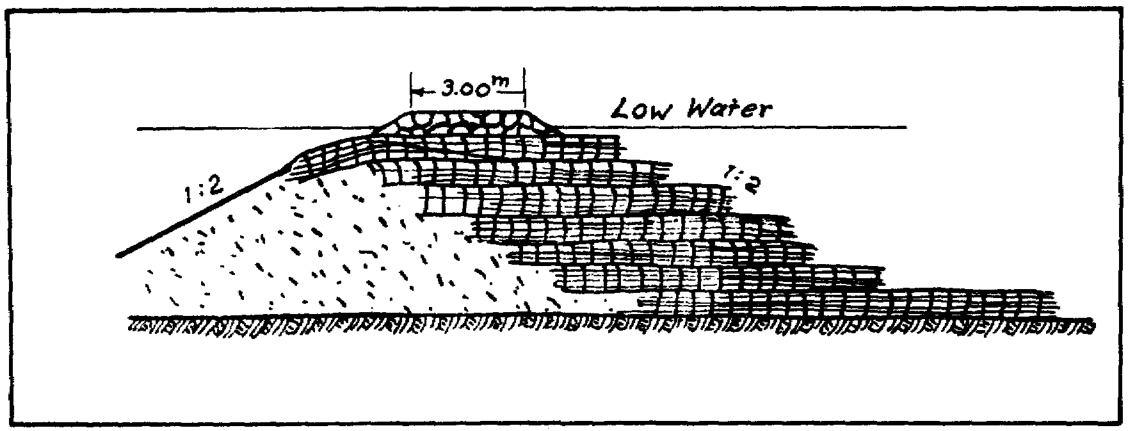


FIG. 48 METHODS OF GROYNE CONSTRUCTION ON THE RHINE

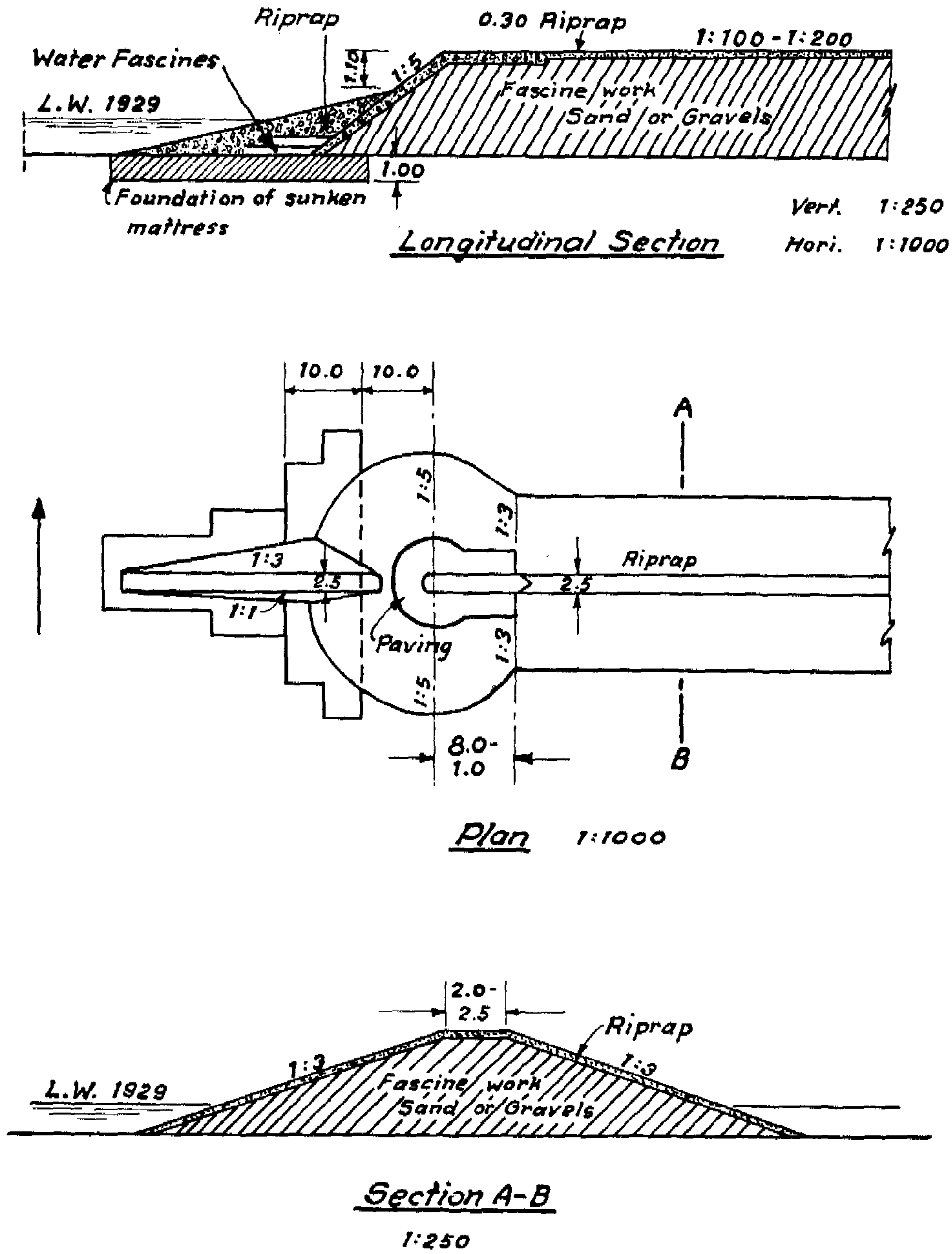


FIG. 49 GROYNE ON THE ELBE WITH HEAD SILL

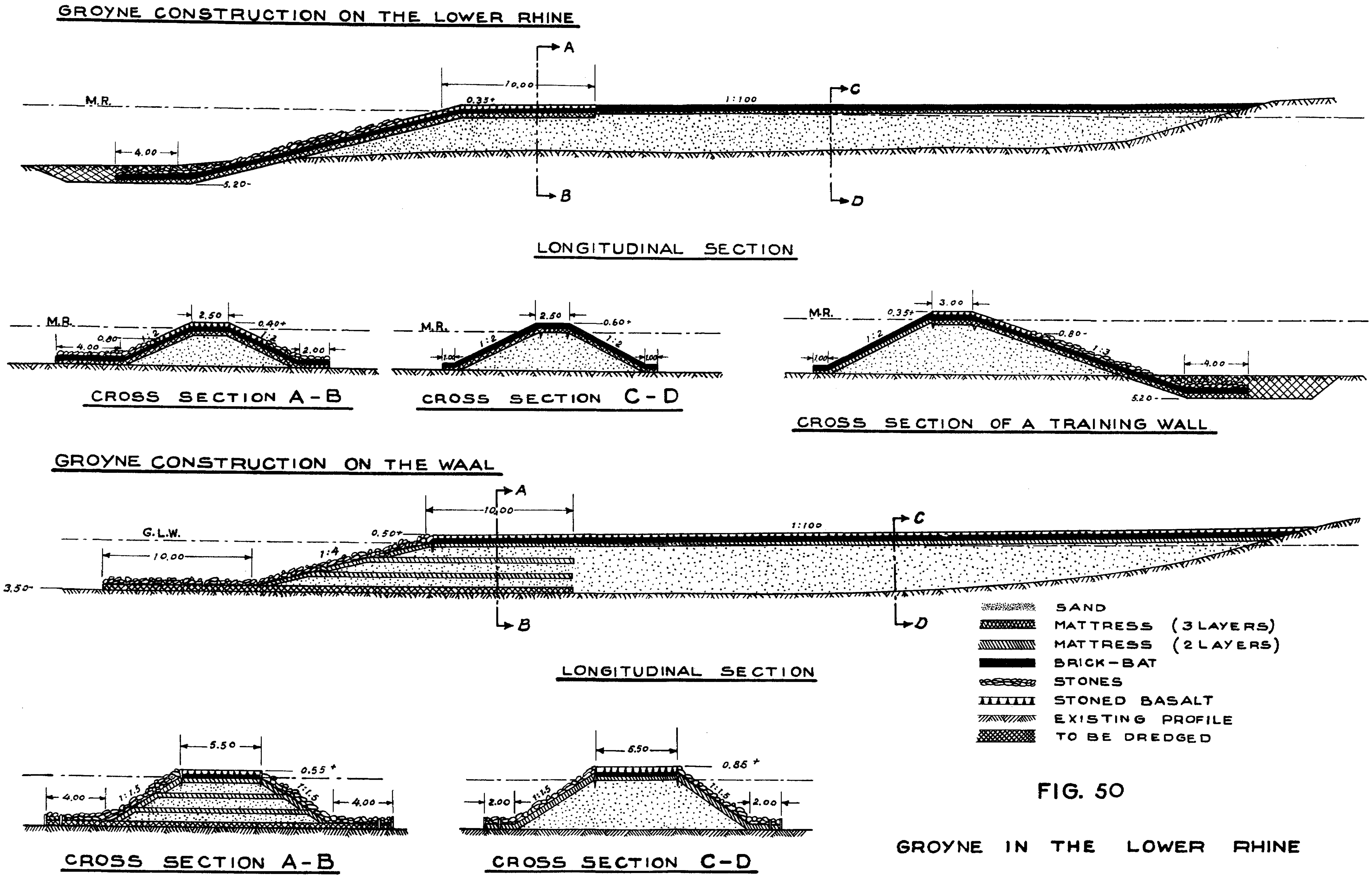


FIG. 50

GROYNE IN THE LOWER RHINE

IN THE NETHERLANDS

SCALE 1:250

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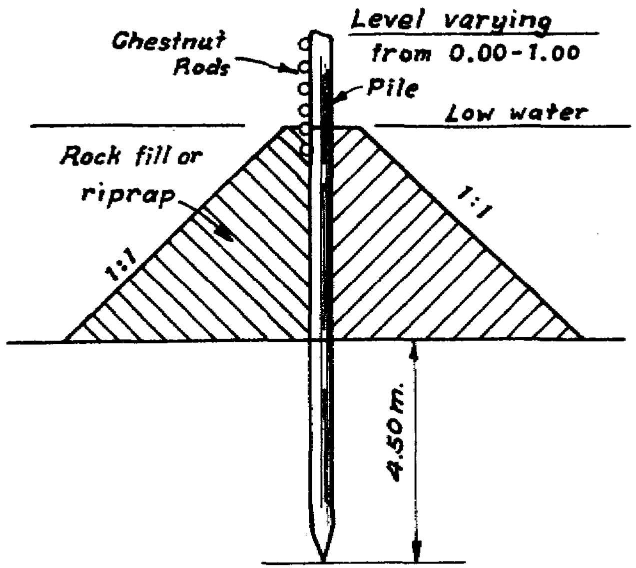

FIG. 51 SECTION OF GROYNE AND TRAINING WALL OF THE LOIRE

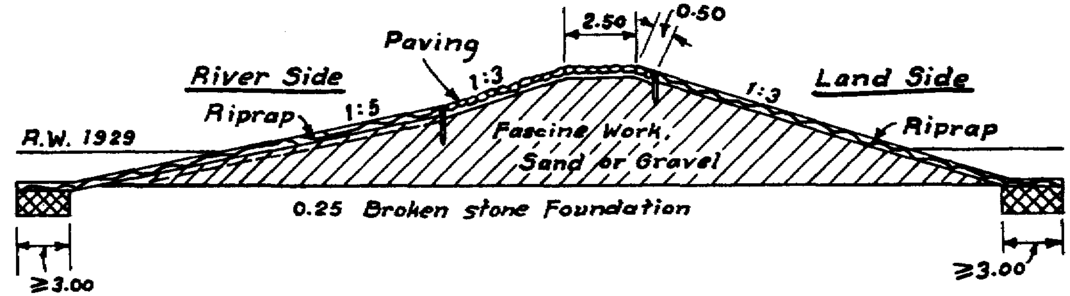

FIG. 53 TRAINING WALL ON THE ELBE

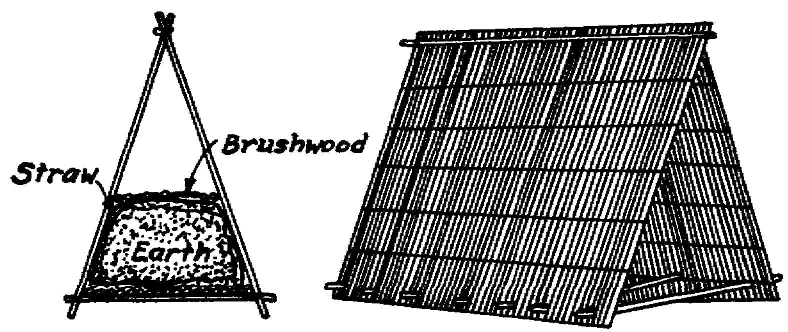

FIG. 52 BRUSHWOOD TENTS

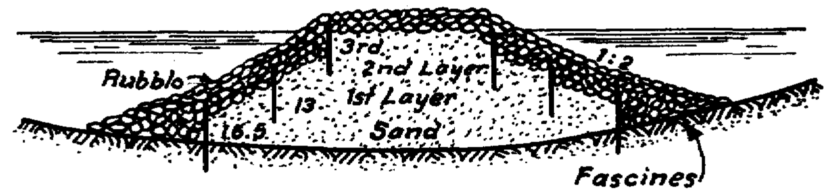

FIG. 54 TRAINING WALL FOR RIVER PO

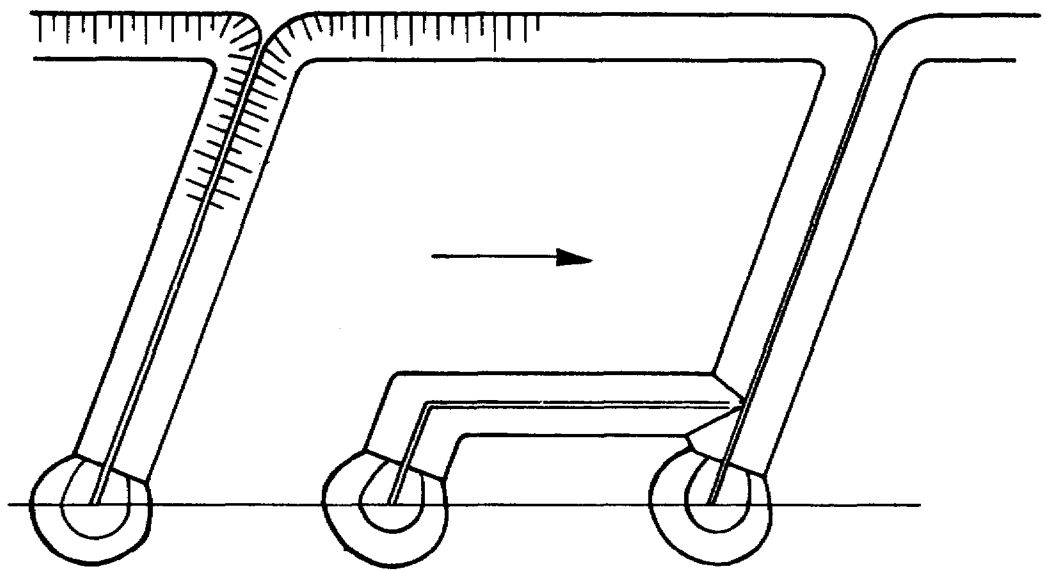

FIG. 55 HOOKED GROYNES

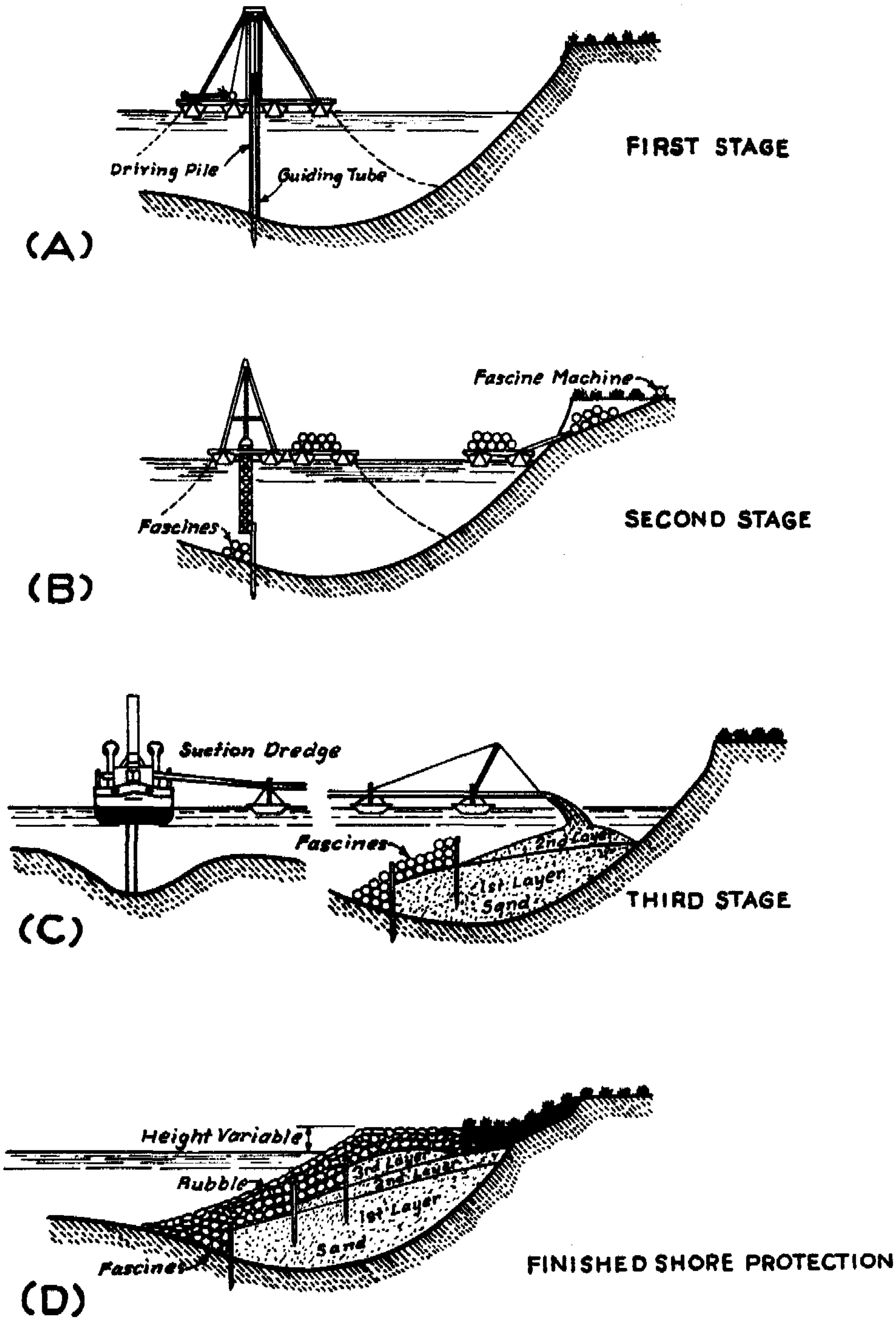

FIG. 56

CONSTRUCTION OF SHORE PROTECTION ON THE RIVER PO

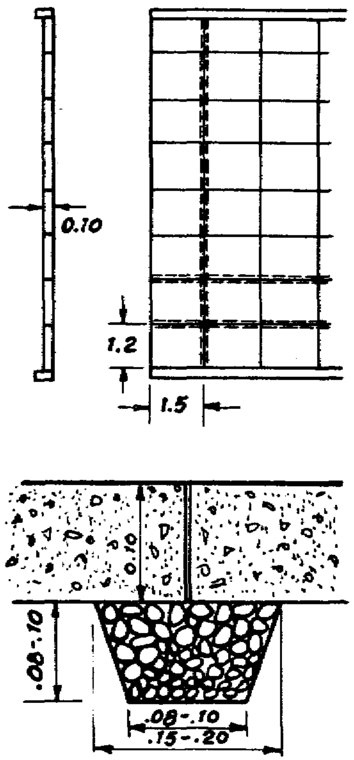

FIG. 57

CONCRETE BLOCK PAVEMENT

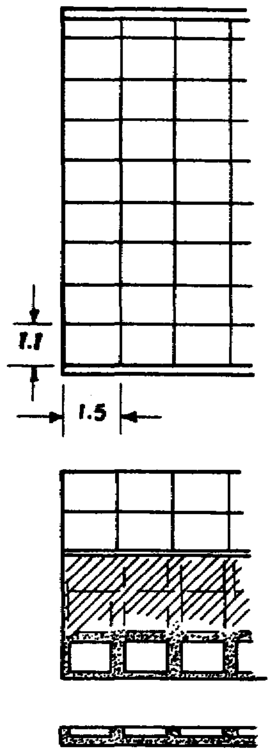

FIG. 58

MELOCCO CONCRETE BLOCK PAVEMENT

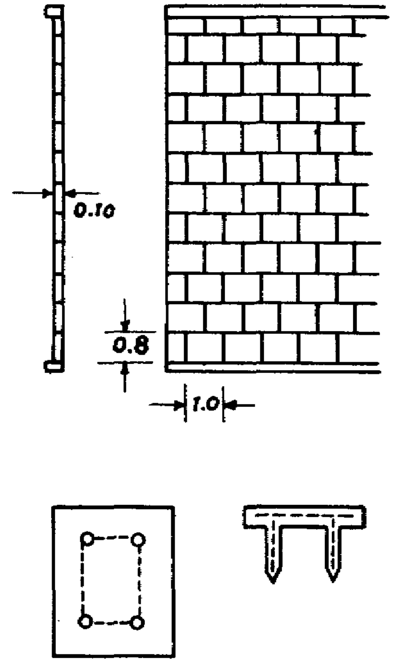

FIG. 59 KISS CONCRETE BLOCK PAVEMENT

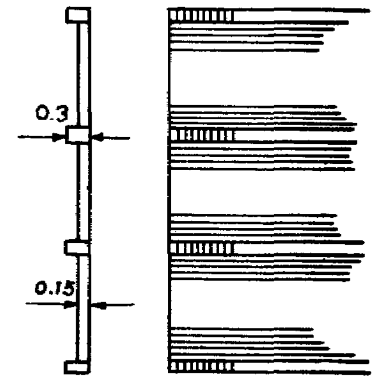

FIG. 60

BRICK PAVEMENT (TYPE I)

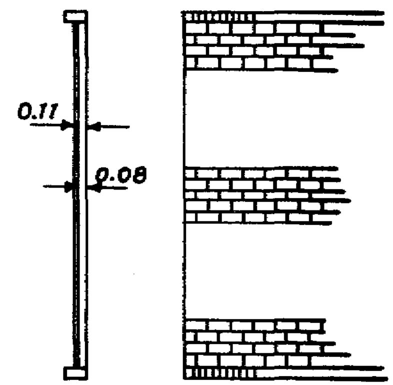

FIG. 61

BRICK PAVEMENT (TYPE II)

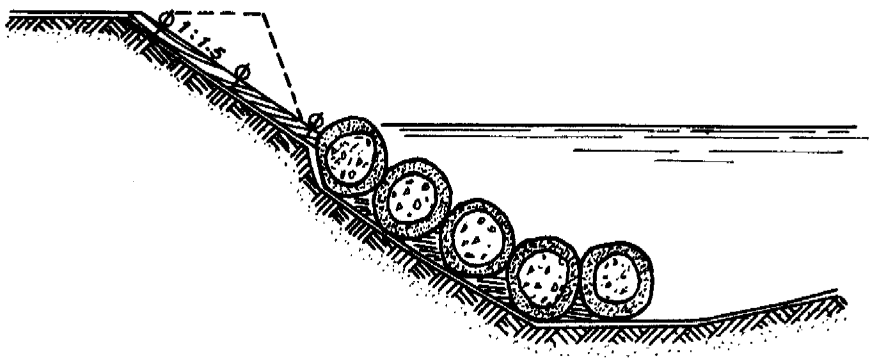

FIG. 62 BRUSH ROLLERS

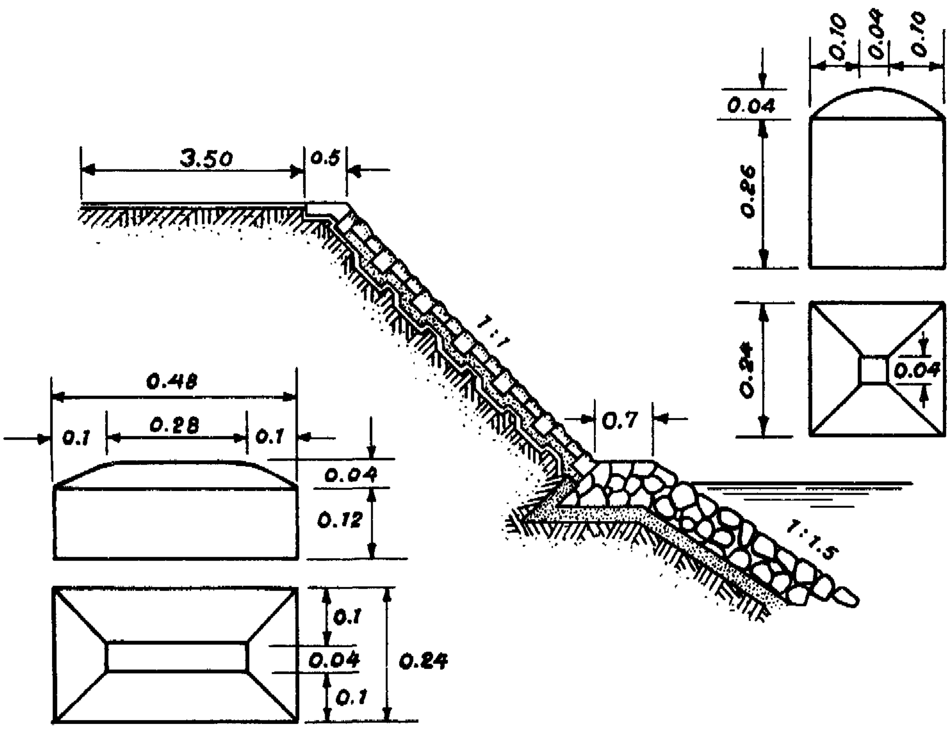

FIG. 63 SQUARE AND RECTANGULAR BLOCKS

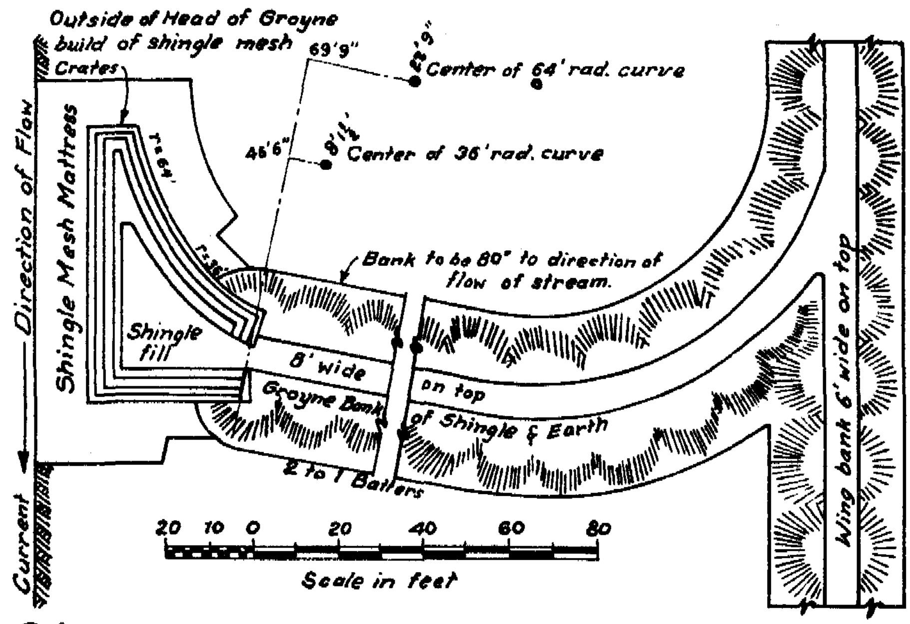

FIG. 64 PLAN OF TYPICAL IMPERMEABLE GROYNE

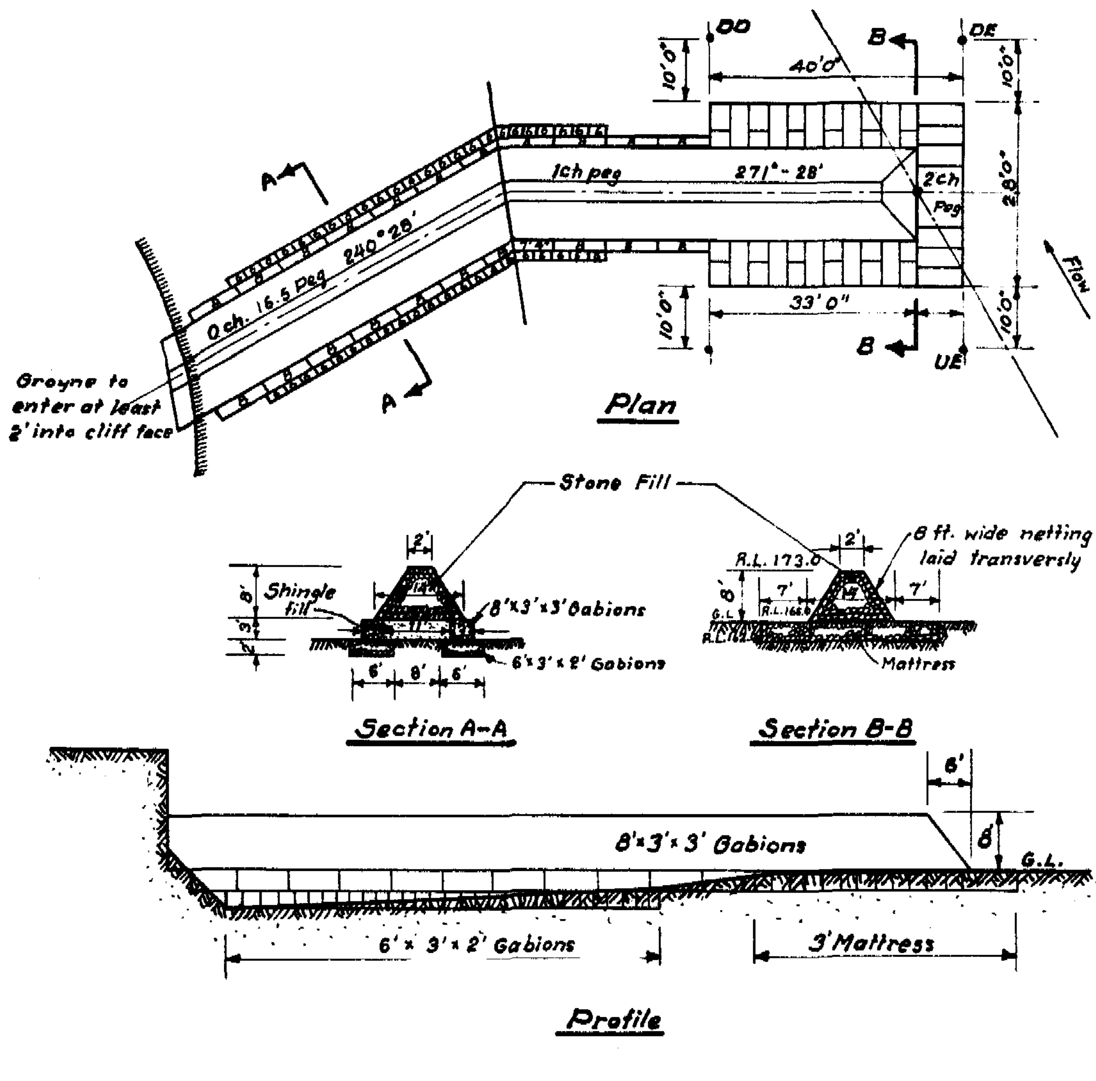

FIG. 65 TYPE OF SHINGLEMESH GROYNE, ASHLY RIVER

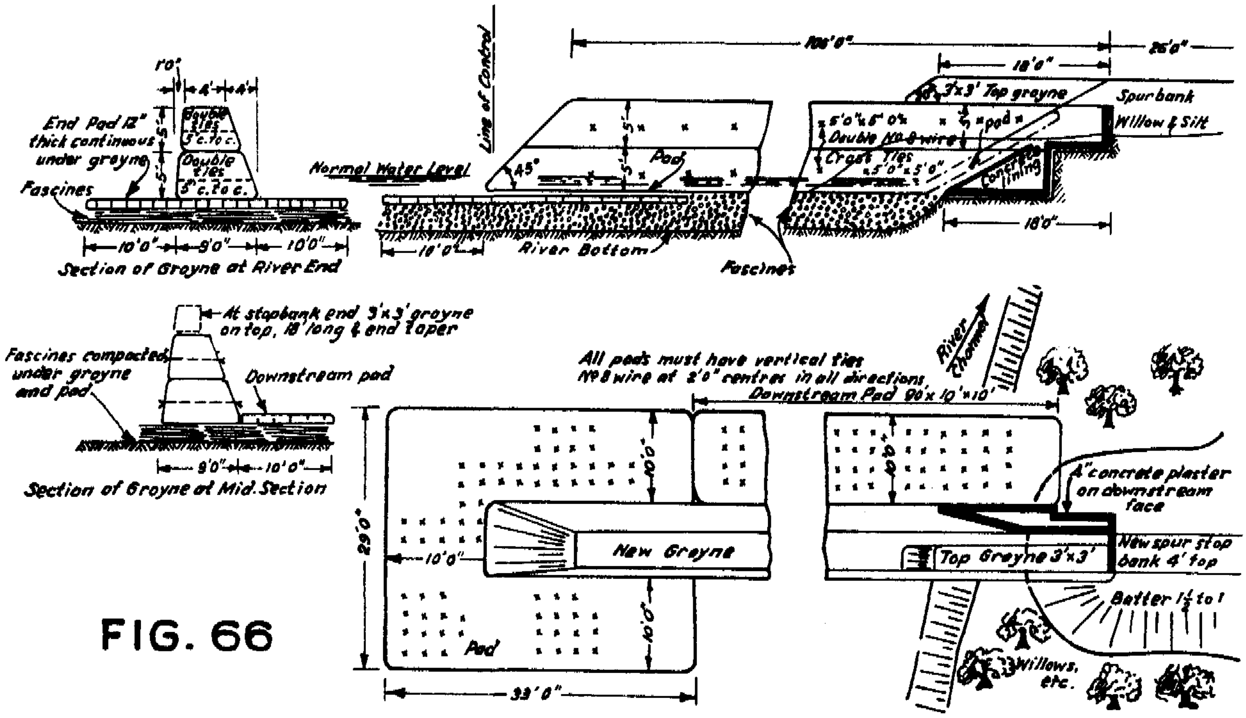

FIG. 66

TYPE OF SHINGLEMESH GROYNE WITH MATTRESS ON DOWNSTREAM SIDE

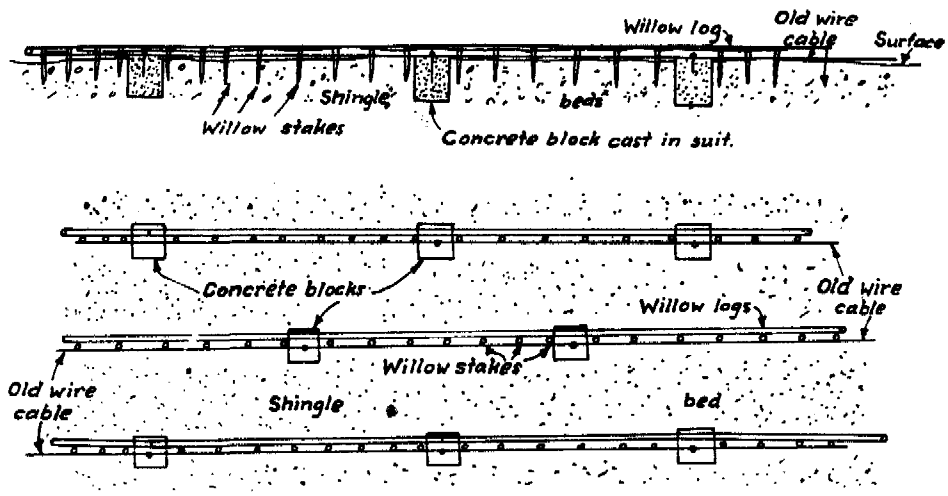

FIG. 67 TYPE OF WILLOW GROYNE

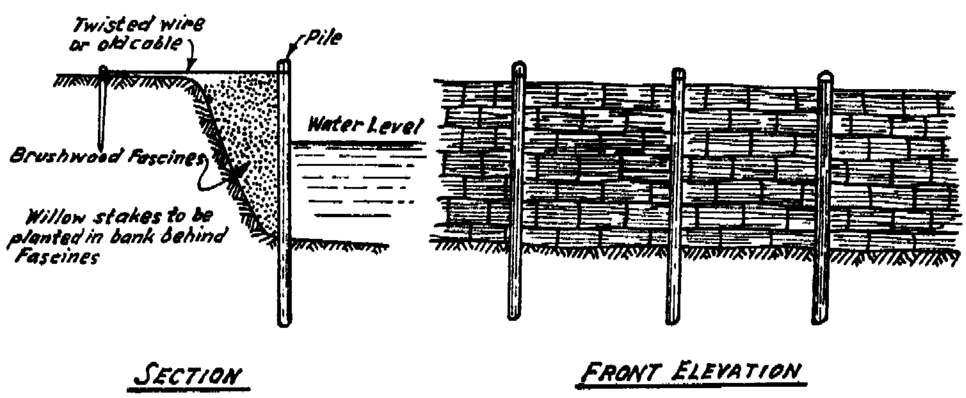

FIG. 68 PILE AND FASCINE REVETMENT

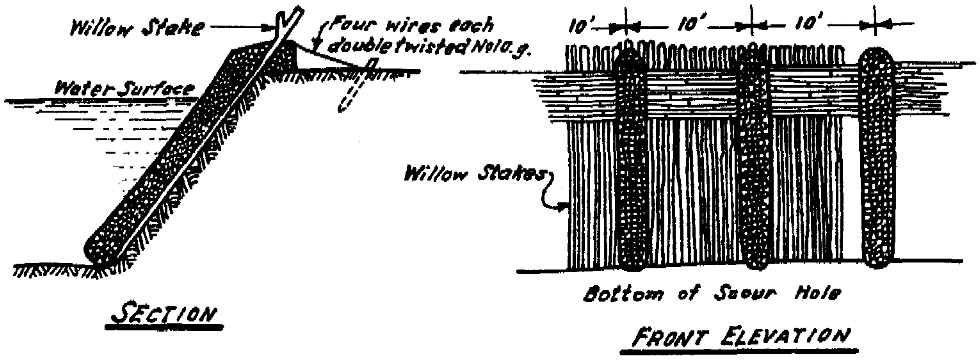

FIG. 69 STONE SAUSAGE AND WILLOW STAKE PROTECTION

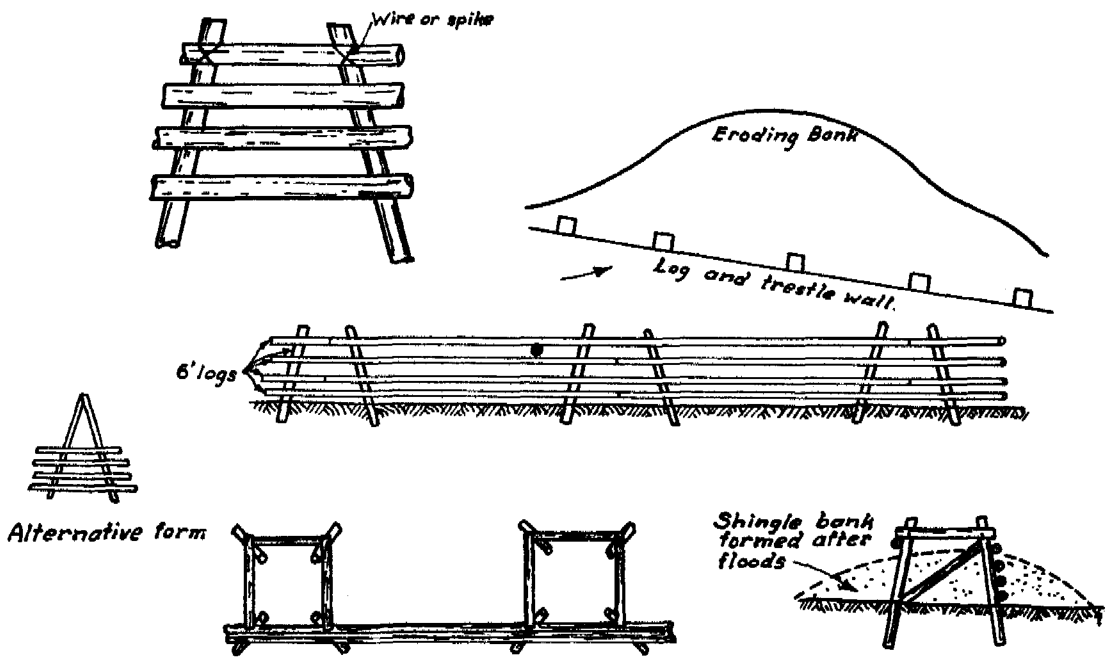

FIG. 70 TYPE OF ROUGH LOG TRAINING WALL FOR PROTECTING ERODING BANK

UNITED NATIONS

FLOOD DAMAGE AND FLOOD CONTROL ACTIVITIES IN ASIA AND THE FAR EAST

This publication, which is the first number of the "Flood-Control Series" prepared by the Bureau of Flood Control of the Economic Commission for Asia and the Far East, consists of 100 pages of text and 18 four-colour maps as well as many other graphs and illustrations. Material for the study has been obtained from technical organizations dealing with flood control within the ECAFE region, in reply to a questionnaire addressed to them by the Bureau.

Chapter I gives a summary hydrology of Asia and the Far East broken down into temperature, pressure and wind, precipitation and cyclone, run-off and silt flow. Chapter II traces the development of flood control measures and physiographical influences thereon. Chapter III lists flood damage and flood control activities for various rivers of the region, including the Brahmaputra, Damodar, Ganga, Kosi and Mahanadi in India, the Hai, Huai, Pearl, Yangtze and Yellow River in China, the Chao Phya in Thailand, the Irrawaddy in Burma, the Kelani in Ceylon, the Solo and Brantas in Indonesia, the Red River and Mekong in Indochina and the Pampanga and Agno in the Philippines. The four-colour maps give details of flood control works for these rivers. A summary of basic data on these rivers is also included.

UN publication, sales No. 1951 II.F.2. Price US$1.50, or equivalent in other currencies. Available in separate English and French editions.

METHODS AND PROBLEMS OF FLOOD CONTROL IN ASIA AND THE FAR EAST

This publication, which is the second number of the "Flood-Control Series" prepared by the Bureau of Flood Control of the Economic Commission for Asia and the Far East, consists of 45 pages of text and 8 diagrams. Material for this study has been collected by experts of the Bureau during their field trips to various countries of the region and from technical literature on the subject published both within and outside the region. The study is divided into two main sections. Section I gives a comprehensive review of flood control methods employed in different countries of the region and section II discusses the various problems arising from different aspects of flood control with particular reference to the characteristics of rivers in the region and the special conditions met with in this part of the world. The problems are discussed under the following main heading: General consideration of stability of rivers; Dikes and river training; Storage, detention and diversion; Soil and water conservation and some other aspects of flood control. A third section gives the conclusions.

UN publication, sales No. 1951 II.F.5. Price US$1.15. Available in separate English and French editions.

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This publication, which is the third number of the "Flood Control Series" prepared by the Bureau of Flood Control of the Economic Commission for Asia and the Far East, comprises the proceedings of the Technical Conferences on Flood Control organized by the United Nations Economic Commission for Asia and the Far East and held at New Delhi, India from 7 to 10 January 1951 and the thirty-two papers presented at this Conference by experts from member and associate member countries and by the various specialized agencies of the United Nations.

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