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To Beam As A Beacon of Knowledge

RIVER TRAINING AND PROTECTION WORKS FOR RAILWAY BRIDGES

November 2016

INDIAN RAILWAYS INSTITUTE OF CIVIL ENGINEERING
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FOREWORD TO FOURTH EDITION

The book “River Training and Protection Works for Railway Bridges” was first published in year 1983.The revised fourth edition has been brought making some changes taking into account the current sub structure code and Indian Railway bridge manual in chapter 5 and 6.

It is hoped that the book will be found useful by the field Engineers, involved in the planning, design & construction of railway bridges.

The suggestions for improvement are welcome.

November 2016

N. C. Sharda
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PREFACE TO FOURTH EDITION

The book “River Training and Protection Works for Railway Bridges” was first published in year 1983. Subsequently, it was revised and enlarged as 2nd edition in 1998 and 3rd edition in 2007.

In this 4th edition following changes have been made -

In chapter 5, the instructions regarding guide bunds, spurs/groyens, approach banks, marginal bunds and closure bunds have been included as contained in IRBM. & in chapter 6 the guidelines regarding estimation of Design Discharge for bridges as contained in Sub-structure Code have been included.

It is expected that it would be useful for railway engineers.

Above all, I am grateful to Shri N. C. Sharda, Director IRICEN for his encouragement and guidance in bringing out the publication.

November 2016

(Ramesh Pinjani)
Sr. Professor-Bridge 2
IRICEN

PREFACE TO THE THIRD REVISED EDITION

The second edition of the book **"River Training and Protection Works for Railway Bridges"** was brought out nine years ago. Keeping in view the popularity of the book among field engineers and to keep them abreast of the latest developments taking place in this field, the third revised and enlarged edition has been brought out by adding new chapters on Remote Sensing, Filters, Permeable Structures and Mathematical Modelling so as to provide latest techniques being used in design of river training and protection works for bridges.

Although every effort has been made to present the book in error free manner, yet if there is any suggestion or discrepancies, kindly do write to us.

Shiv Kumar
Director
IRICEN

ACKNOWLEDGEMENT TO THE THIRD REVISED EDITION

The first edition of the book on “River Training and Protection Works for Railway Bridges” was published in 1983. It was written by Dr. S. V. Chitale, retired joint Director of Central Water and Power Research Station, Khadakwasla, Pune. It was extremely popular amongst field engineers. The second revised edition of this book was brought out in 1998 by making some changes necessitated due to changes in Substructure and Foundation code.

A need was being felt to revise the book keeping in view of rapid developments taking place in the field of River Training and Protection Works, especially in the areas where latest technology, viz. remote sensing, geo-fabric filters can be used extensively and effectively for designing river training and protection works. Age old technique of channelling flow of rivers with the help of permeable structures has been added as a separate chapter. In this revised & enlarged edition, a new chapter on Mathematical Modelling has been added. In addition, a number of actual colour photographs have been incorporated in the book for better understanding of the subject.

The revised edition has been brought out with a view to provide maximum information to users. In this effort, Shri R. A. Oak, retired Chief Research Officer, Central Water & Power Research Station, Khadakwasla, Pune has provided invaluable assistance by penning new chapters, I thank him for his valuable contribution. I also thank Shri Ganesh Srinivasan for providing assistance in word processing.

Above all, I am grateful to Shri Shiv Kumar, Director, IRICEN for his encouragement and valuable guidance in bringing out this publication.

Ajay Goyal
Senior Professor/Bridges
IRICEN

PREFACE TO THE SECOND REVISED EDITION

River training and protection works for railway bridges play a major role during planning, construction and service of these bridges. The design of various components of training and protection works was developed over a period of about 100 years and the present methods are time tested. The first edition of this book was published in 1983 and was very popular amongst Railway Engineers.

This revised second edition has been brought out making some changes taking into account the current Substructures and Foundations Code. The utility of the book has been enhanced by improving the presentation of text matter and sketches. I hope this book will be very useful to Railway Engineers in design, construction and maintenance of river training and protection works for railway bridges.

March 1998

Director
Indian Railways Institute of
Civil Engineering,
Pune

PREFACE TO THE FIRST EDITION

Hydraulic design of a bridge comprises several aspects such as design discharge, waterway, constriction by approaches, guide bunds, protection works, etc. Literature available on these topics is mostly scattered and few publications have attempted to bring important relevant information at one place. The classic work of Spring (1903) 'River Training and Control on Guide Bank System' and another noteworthy contribution made by Gales (1938) provide the designer with authoritative guide but substantial new information has become available subsequently. 'River Training and Control of Bridges' by Sethi (1960) is more recent and brings together views of Spring; Gales and others along with his own but still warrants updating. 'Behavior and control of Rivers and Canals' by Inglis (1956 - Revised 1971) and 'Manual on River Behavior, Control and Training' are two more outstanding treatises of great importance but encompass a very wide range of allied subjects and a bridge engineer has to wade through the entire work to dig out the required bit of information. A cogent and coordinated treatment of all topics of interest to a bridge engineer in the light of present day knowledge on the subject is long felt wanting. Moreover many of the publications cited before are now out of print and stock. The present work strives to fill in this lacuna.

The subject matter is presented in a logical sequence starting with basic background information about types of rivers and their characteristics contained in the first two chapters. Some essential topics like velocity distribution, flow formulae, afflux, scour, parametric relationship evolved by Lacey, are dealt with in the next chapter under 'Principles of River Training'. The next chapter entitled "Types of Training and Protection Works" covers subheads concerning guide bunds, bridge approaches, their protection by spurs and revetment and pier protection aspects in general. Concise design norms for these very components are presented in the subsequent chapter. The last two chapters touch upon the subjects of

hydraulic modeling and measurement of river water levels and discharges. The entire subject matter is thus grouped under the above seven chapters.

Dr. S. V. Chitale, retired Joint Director of Central Water and Power Research Station, Khadakwasla, Pune has written this monogram, which is being published by the Indian Railways Institute of Advanced Track Technology, Pune. It is hoped that this will be useful to all Railway Engineers and other Engineers who have to design, construct, inspect and maintain River Training and Protection works for bridges.

Principal
Indian Railways Institute of
Advanced Track Technology, Pune

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Chapter 1

TYPES OF RIVERS

In Bridge Engineering, intimate knowledge of the environmental medium is as important as that of the structure since their interaction governs safety of the bridges and final regime of the river.

For studying the characteristics and behaviour of rivers, some classification is required to be followed. Basis for such classification could be varied, as for instance, according to the -

- Nature of bed and bank material
- River flood hydrograph,
- Geological age of river,
- Overall river characteristics, etc.

Out of these different classification schemes, the last one is preferred, being the most convenient and appropriate in the context of the subject of bridge hydraulics. In the same river, characteristics go on changing from the head reach in the hills towards the tail end at the outfall into the sea. Suitable division of a river system can, therefore, be made as below :-

(a) Upper (upland hilly) reaches
(b) Submontane (foothill) reaches,
(c) Flood plain reaches,
(d) Tidal reaches, and
(e) Other types.

These are discussed below :

1.1 UPPER REACHES

In the hilly upland reach, the river course and its plan form are mostly governed by hill contours. Rocky banks impose side constraint in lateral movement of the river. The riverbed comprises boulders, shingle, gravel and material of

Photo 1.1 : River Ravi (left) and River Pubbar (right) in the hilly region of Himachal Pradesh. Note : Banks of the river are fully governed by the hills. The bed material is comprised of boulders of various sizes.

smaller sizes. Such heterogeneous material cannot develop bed form of ripples, dunes or waves. River gorge is often deep and narrow and river slope very steep resulting in formation of frequent rapids. River velocity is high and capable of transporting big boulders, which can cause rapid abrasion and damage to structural components such as bridge piers due to friction and impact. Photo 1.1 shows River Ravi and River Pubber in the hilly regions of Himachal Pradesh.

Floods in hilly terrain are generally flashy. Because of landslides, damming up of river flow can occur at times. When such a dam bursts because of overtopping, a sudden flood wave of catastrophic magnitude can sweep down the river. Sutlej River in Himachal Pradesh experienced such catastrophic floods in August / September 2000, due to dam burst in Tibet. It was reported that about 60 bridges located on the river were lost in that catastrophe. Photo 1.2 shows remnants of a suspension bridge lost during the catastrophe.

Photo 1.2 : Remnants of a suspension bridge lost during the catastrophe

The sediment transport in upland reach comprises both heavy bed load as well as suspended load. The velocities being high, sediment transporting capacity and hence the sediment load is very heavy.

In boulder bedded hilly torrents, the channel may at times form deposits of bed material during heavy floods due to soil erosion and land slips. Measures for controlling soil erosion, improving stability of side slopes and arresting bed load are found useful under these conditions. Preservation of soil cover can be promoted by afforestation, gully and check dams, contour bunding etc. For improving stability of side slopes, provision of longitudinal and lateral drainage, breast and toe walls, chutes and in extreme cases bored piles are found effective. Roads running along upper reaches of some of the tributaries of the Ganga and the Brahmaputra have been maintained by providing such elaborate drainage systems. Excessive bed load, which may lead to choking of the channel, can be arrested by construction of debris dams and detention basins. Los Angeles, city in the U.S.A. is protected from boulders rolling down steep adjoining streams by means of debris dams. On Gandak River in the Himalayan gorge, an extensive widening of the valley floor has provided the river with a natural detention basin where coarse bed load is arrested.

1.2 SUBMONTANE REACHES

The foothill area in between the high mountains and relatively flat flood plain is termed submontane region. Local names such as Bhabar area are also in vogue. The river in submontane reach suddenly acquires much flatter bed gradient than the hilly mountainous tract. The flow velocity, therefore, drastically reduces and so its sediment transporting capacity. This phenomenon encourages deposition of the excess sediment load. The rivers in the reach are normally prone to aggradation, which in turn leads to lateral shifting of the course. An alluvial fan, also termed as inland delta is built up and progressively it is raised and extended. Bed and bank material is not much different except at the extremities of the delta. The bed gradient is not as steep as in the gorge but not as flat as in the plain. Relative to lower reaches, the river course being steeper has got higher velocities. These characteristics develop with several active channels existing simultaneously intersecting each other and forming islands in between. All these channels normally over-flow during high flood and the river acquire a very wide and shallow cross section. Foothill reaches of the tributaries of the Ganga and Brahmaputra river

Photo 1.3 : Tributary of River Sutlej in Himachal Pradesh. Note : The wide range of sediment sizes, deposited on the riverbed. Existence of old dry channel along the right and left banks and existing channel in the middle indicates instability of the river channel.

systems are illustrative of braiding pattern. Photo 1.3 shows a tributary of River Sutlej in Himachal Pradesh. Peak floods can more often be flashy. Intensity of suspended and bed load transport is intermediate between upland and flood plain reaches.

1.3 FLOOD PLAIN REACHES

The river in this reach has a relatively deep and narrow cross section with medium slope. In high floods, extensive heavy spills can occur. These spills result in deposition on over bank area of suspended sediment having high fertilising value. Flooding due to such river is thus beneficial in increasing agricultural productivity. On the other hand, frequent and prolonged inundation can result in untold miseries and hardships to the population and sometimes causing enormous loss of property and even life.

The channel is usually of meandering pattern with meanders at times moving down-stream or forming cut offs across the neck. The Khadir width within which all meanders are contained is several times bigger than the river width at bankful stage though during high floods, the entire width of the Khadir is occupied. The river bank material contains a good percentage of clayey silt and the side slope can therefore be steep.

The bed is normally sandy and during floods big sized bed forms can develop like sand dunes and waves, which move progressively downstream. The floods are sustained and of relatively long duration as in this reach, several tributaries may join the parent river and form the river system.

Sediment concentration is high for fine as well as medium and coarser sizes. Bed load in comparison with suspended load is much smaller.

1.3.1 Stable type

Rivers in flood plains are normally stable with no perceptible rise or lowering of the riverbed occurring from year

to year over long periods. Changes in the river bed do occur from season to season, but on an annual basis, bed level changes are nominal. Ganga River is a notable example of such type of rivers.

1.3.2 Aggrading type

On the other hand, some rivers are known to aggrade and progressively raise their beds by sediment deposition. In the process, overbank spills increase year after year until, with occurrence of abnormal floods any year, an avulsion can take place. The Yellow River in China has been notorious for such avulsions causing extensive devastation in its wake. Aggradation can be the result either of increased sediment yield or reduction in flood discharge or both these factors.

1.3.3 Degrading type

Opposed to aggrading tendency, the river may be prone to degrade its bed, if sediment supply reduces or flood discharge increases. Reduction of sediment supply can be due to reforestation and land management in the catchment or because of construction of a reservoir impounding appreciable quantity of water and the sediment. Colorado river in the U.S.A downstream of Hoover dam, which was previously known as Boulder dam, is a well-known example of degrading river. The river bed scoured by as much as 5.5 m at about 16 km in the first 7 years of completion of the dam. This lowering reduced progressively in the downstream direction, the length affected by degradation being as long as 155 km.

Aggradation reduces available waterway area for passage of flood flow and causes progressive rise in flood levels. On the other hand, degradation can affect safety of foundations. Thus, both aggrading and degrading rivers affect utility and threaten safety of the structures.

1.4 TIDAL REACHES

At the outfall end of the river system, the effect of

sea tides is felt and predominates over the tidal reach. The tidal effect depends on tidal range, which is the difference between high water and low water and varies from one tidal waterway to another. The effect also changes from day to day due to changing position of the earth with respect to sun and the moon. On full moon and new moon days, lunar effect and so the tidal ranges are the highest. These tides are called *spring tides.* On the other hand, at quarters lunar effect is the least and so also the tidal range. These are called *neap tides.* When moon is the nearest, the tide is called *perigee tide* and when the farthest it is called *apogee tide.* As the flood tide progresses from the sea into the estuary and tidal creeks, shallow water effect is felt and wave form gets distorted, the rising limb of the tidal wave becoming steeper. Accordingly, the wave velocity becomes faster and in an extreme case, the flood tide moves up like a wall when it is called a *bore tide.*

Along with changes in water levels, the tides also cause river waters to become saline. The maximum salinity is at sea end and progressively reduces towards the upstream end of a tidal reach. The salinity changes along the length and according to tidal effect. In addition the fresh discharge of the river exerts influence on both the tides and salinity.

Due to salinity and consequent flocculation, fine suspended sediment deposits in tidal waterways. Deposition process is also affected by changes in density of water. The river regime is thus markedly governed by tides and obstruction caused by a bridge to propagation of tides can seriously affect the river regime.

1.5 OTHER TYPES

1.5.1 Flashy Rivers

The rivers in upper hilly reaches and submontane area often experience flashy floods. Heavy storms can cover and concentrate over relatively smaller catchments of such rivers. A flood wave of short duration but of high intensity then rushes down the river like a flash. In such a flood, heavy

sediment load can be brought down by the river, which can be deposited on the way. Flashy floods cannot scour the riverbed sufficiently at constrictions within guide bunds thereby causing high afflux. In case of Luni River in Rajasthan, a flashy flood in 1944 resulted in rapid rise of water level of 24m in 2 days. In case of the Bandy, its tributary, flood level rose even faster by 48 m in the same period. The resulting afflux upstream of the bridge was about 1.83 m, which caused breaches in left approach and damage to three end piers on the left flank.

Land slides some times cause natural damming. When such dams burst, a devastating flood wave can be generated as in case of Alaknanda River in July 1979. The example of Sutlej River in August 2000 in Himachal Pradesh has already been cited as recent example of such floods. Photo 1.2 shows a damaged suspended bridge on river Ravi located about 25 km downstream of Rampur town, during the same flood. The suspension wires and abutments only are left out after the floods. The normal procedure of arriving at the design flood for a rivarian structure cannot cater for such contingency. Such a possibility cannot, however, be completely ignored. Suitable increase in design discharge may, therefore, be made based on past experience of natural dam bursts caused by landslides in the catchment under consideration or adjoining catchments.

1.5.2 Virgin Rivers

Some of the rivers flowing in arid regions of Rajasthan and Kutchh have no outfall in the sea nor do they join any other stream. Such rivers, after traversing some distance, lose all their waters by percolation and evaporation. Rivers of this type are called Virgin Rivers. The example of three main rivers of Kutchh can be cited for this purpose.

Rivers Banas, Khari-II and Saraswati originate in Rajasthan, and travel through Kutchh. Normally, these rivers experience very little or no flows even in monsoon season. However, occasionally the rivers experience very flashy floods. Though, Narmada Main Canal crossings for these rivers have high design discharge (28615 cum/s, 9101 cum/s,

and 11862 cum/s respectively), most of the discharge is known to vanish by the time they reach to the Rann of Kutchh. In fact, the rivers are nearly non-existent within a short distance of 20-25 km from the Canal Crossings.

1.5.3 Himalayan Rivers

Himalayas are geologically young and the nature of rock formation is of sedimentary origin. Intensity of rainfall is heavy. The region is susceptible to severe earthquake shocks in addition. All these factors are responsible for big and numerous landslides and enormous yield of sediment from the catchments of Himalayan rivers. Thus, torrential floods and heavy sediment load are the characteristics of Himalayan Rivers. In addition to monsoon floods considerable snow melt discharge makes these rivers carry perennial flow.

After debouching into the plains, all Himalayan Rivers join the Indus, the Ganga or the Brahmaputra river systems. These river basins are formed of deep alluvial deposits and hence all watercourses in these basins belong to the category of alluvial streams.

1.5.4 South Indian Rivers

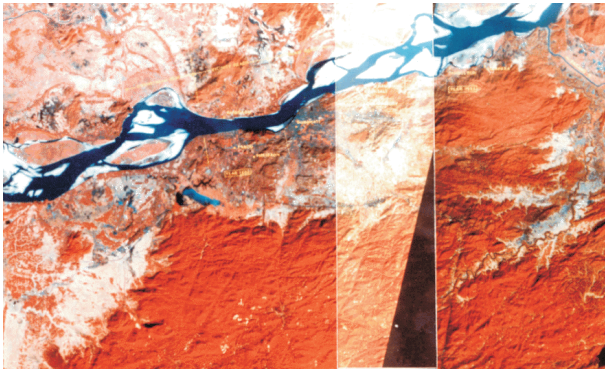
In contrast to Himalayan geology, the South Indian peninsula is geologically much older and formed of igneous rock formations except at the outfalls of the rivers into the sea where Mahanadi, Godavari, Krishna and Cavery deltas are formed. Sediment yield and concentration are accordingly smaller. These rivers belong to the category *of incised rivers.* The term ‘incised’ characterises channels generally formed in the process of degradation. The bed and bank material is, therefore, different from the sediment transported in such rivers most of which comes from the catchment due to denudation and soil erosion. The bed and banks of the river are usually highly resistant to erosion. The river systems in South India are older and more stable. Hence, tendency for shifting of river courses and for aggradation or degradation is insignificant.

■ ■ ■

Chapter 2

BEHAVIOUR OF RIVERS

A bridge engineer needs to be well conversant with the river behaviour, which is largely influenced by the form or shape of the river alignment in plan. Plan form of a river is designated by the term “Channel pattern” which can be either meandering, braided or straight.



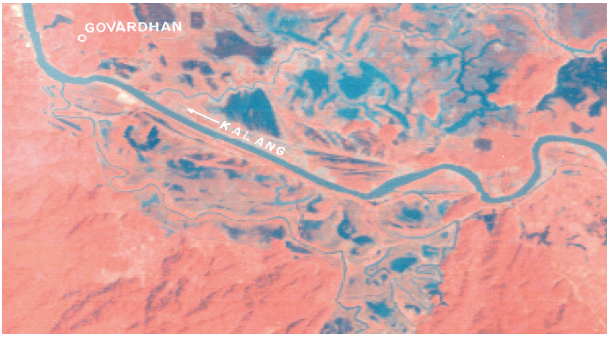


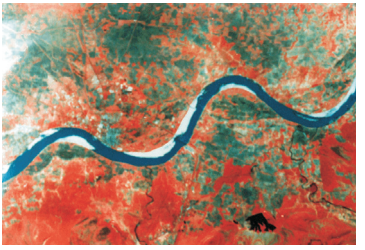
Photo 2.1 (A) : (Top) Showing Brahmaputra River at Guwahati. Straight course of the river is due to hillocks located on both banks. (Bottom) Showing straight chhanel of Kalang River, Assam

Straight courses of rivers are not met with very frequently. Leopold and Walman [2.1] have observed that they are more of an exception, the straight lengths not obtaining for more than ten times their widths.

Photo 2.1(A) shows more or less straight river courses. In one case, it is due to the movement of channel restricted by the hills. In the second case, the straight length is natural. However, it can be seen that it might be a temporary phase, supporting the above statement made by Leopold and Walman.

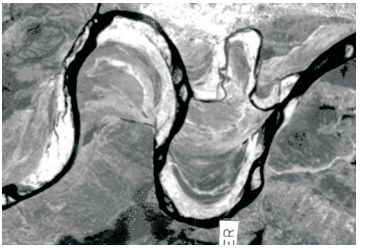
2.1 MEANDERING PATTERN

River courses normally follow tortuous alignment. Such curvilinear courses are termed meanders. River meanders are rarely regular and uniform in shape. The different shapes frequently found in nature are — (i) Normal (Sinusoidal) (ii) flat (iii) acute or intense or sharp, etc. Some of these are shown in Photo 2.1 (B).

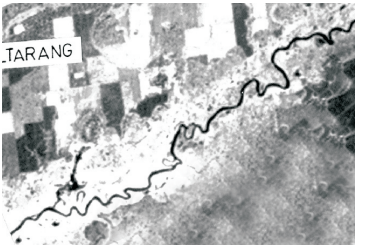


Regular meanders-Narmada River at Hoshingabad, M.P.

Regular flat meanders-Dangori River,Assam



Regular sharp meanders-Narmada River at Hoshingabad, M.P.



Regular and intense meanders-Dangori River,Assam

Photo 2.1 (B) : Examples of river meanders

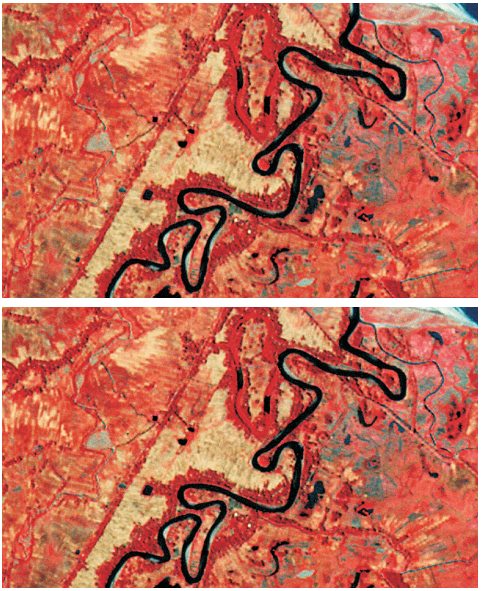


Photo 2.1 (C) : Examples of complex type of river channel

Some complex type of river meanders are shown in Photo 2.1(C)

Some times the meanders change their shape or move downstream. Such meander characteristics have important bearing on the performance and safety of bridges and hence warrant study.

The topic of remote sensing has been discussed in the next chapter, the same may be referred to for further understanding. However, the reader might not feel any problem in understanding the different shapes of the river channels as seen from the imageries.

2.1.1 Meander characteristics and causative factors

Meanders have been attributed to different factors by different investigators. The more recent and widely accepted theory is that formation of a meandering course is nature's way of minimizing variance of river parameters and the rate of expenditure of energy. [2.2] [2.3]

Some meanders are very flat while others very sharp or acute. These and all possible shapes in between can be depicted schematically using arc of a circle as shown in Fig. 2.2. River meanders are rarely so regular and uniform in shape. The conceptual model using circular arc, however help in quantitative assessment of the behaviour of meanders. Definition sketch giving symbols is presented in Fig. 2.3.

Shape of the meanders can be defined by the ratio of length along the river to straight length along the valley, LR/LV. For understanding how shapes of meanders with different values of LR/LV can be developed. It is helpful to study the process of bend erosion. In bend flow super elevation is obtained along the concave bank which results in formation of secondary transverse flow. Effect of this transverse flow is to shift the zone of high velocity and maximum depth to the concave side and render the side slope of the bank stipper. The river bank then becomes unstable and fails and progressively recedes.

When the river section is narrow and deep, shifting of the zone of higher velocity and bigger depth towards the concave bank occurs earlier within a small bend angle and near the apex of the bend. In the process, the meander curvature accentuates. On the other hand, in a river with wide and shallow section, the shift of high velocity and big depth of flow to the concave bank requires longer travel distance and hence the location of maximum depth and steepest side slope in a bend occurs downstream of the apex point. Bank erosion at this location makes the curvature flatter. Thus wider and shallower the cross section of the river, the meander curvature becomes flatter. Meanders of different tortuosities thus appear to be generated on account of differing cross sectional shapes of the rivers.

2.1.2 Cut offs and movement of meanders

Because of progressive bend erosion in narrow and deep rivers, the meander shape changes but not the location. With continued erosion, a cut off occurs across the neck of a hairpin bend as shown in Photo 2.4. The original course of the river then turns in to a lake. Due to its typical shape, these lakes are called Oxbow lakes. An example of such lakes is also shown in Photo 2.4.

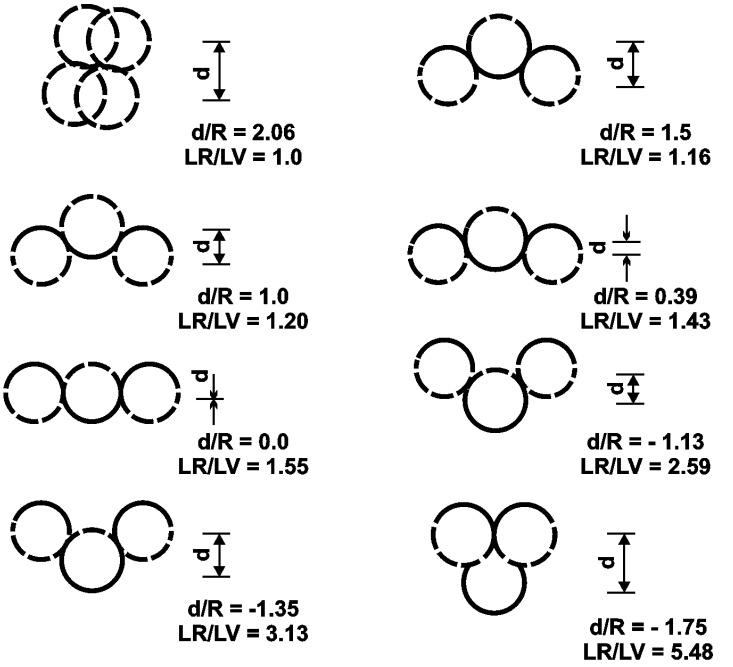


Fig. 2.2: Meander shapes developed using circular arcs

d = VERTICAL DISTANCE BETWEEN CENTRES OF SUCESSIVE CIRCLES, R = RADIUS OF THE CIRCLES, LR = LENGTH ALONG THE RIVER AS IN FIG. 2.3, LV = LENGTH ALONG THE VALLEY AS IN FIG. 2.3

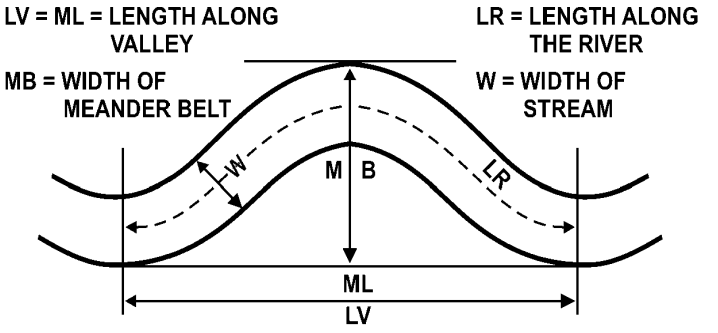


Fig. 2.3: Definition sketch for meander dimensions

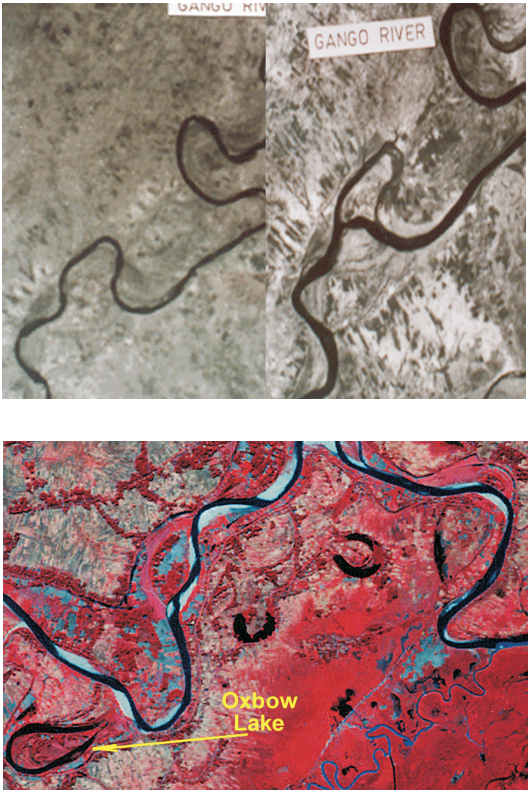


Photo 2.4 : Showing development of cut-off observed in Gango River, Assam and formation of Oxbow lakes

Relations have been evolved to arrive at the likely ratio of (LR / LV) beyond which a cutoff is likely to develop. However, it may be added here that such relations should be taken as only tentative. There can be many other factors, which could act against the development of cutoff. Photo 2.1(C) is one of such examples where the cutoff has not developed.

Natural cut off increases the river slope and accelerates the velocities in the upstream reach and thus brings about

formation of more cutoffs in its wake. On the other hand, in the case of wide and shallow rivers, the meander shape does not change but the whole meander train moves downstream. This downstream travel of meanders results in cyclic changes of the meander position within Khadir at any fixed location. In the case of Ganga river at Mokamah and Mansi the meander was known to occupy the same position either at the south or at north edge of Khadir after a period of about 70 years as shown in Fig. 2.5.

Behaviour of meanders with flat and acute bends being different, it is important to know the sinuocity 'LR/LV' associated with these two shapes. For semi circular shapes, value of LR/LV is generally less than 1.57, while in case of acute bends, LR/LV is normally more than 1.57.

2.1.3 Meander relationships

Bigger the discharge, the meander length and belt may be expected to be also bigger. From Lacey equation, $P = 4.836\ Q^{1/2}$, it is logical to presume that the parameter discharge 'Q' in cumecs can be expressed in terms of the parameter width 'W' in m which nearly equals the wetted perimeter 'P' in m in case of rivers. Further more, when ML and MB are individually related to Q, the ratio ML/MB should also permit quantitative relationship. Working on such reasoning, various investigators have attempted and evolved meander relationships. In India the Inglis relationships are widely known. Schumm, Carlston, Leopold and others in the USA, and by Ackers and Charlton in U.K have also derived meander relationships. It was seen before that in case of rivers with wide and shallow cross sections, the meanders achieve flatter curvature and vice versa. It was also observed that narrow and deep rivers have generally associated characteristics of steeper slope and coarser bed material. A relationship for tortuosities can, therefore, be developed between these relevant parameters[2.4]. All relationships mentioned above are presented in Table 2.1[2.5].

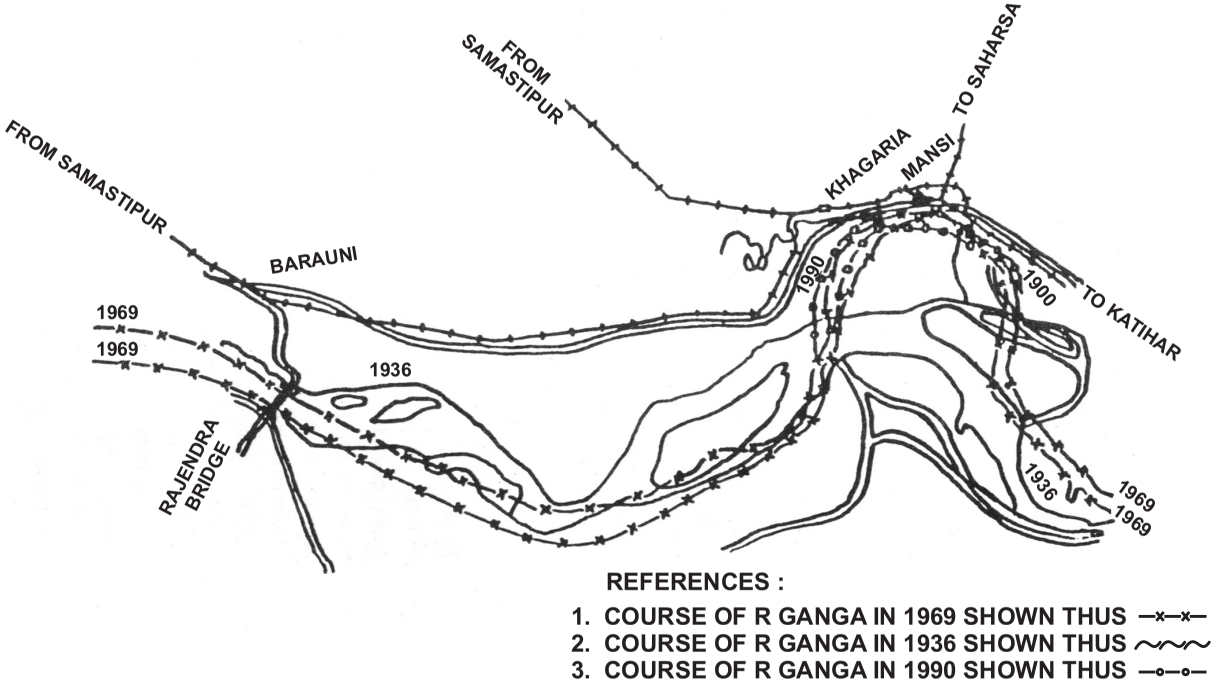


Fig.2.5 :Cycle changes in Ganga river at Mansi

Table 2.1

Sr.No.	Investigator	Year	Relationship	Data	Remarks
1.	Ferguson	1863	ML = 6W	Ganga	
2.	Jefferson	1902	MB = 17.6 W	American and European Rivers	
3.	Inglis	1939	$ML = 49.63Q^{1/2}$	Shaw's data of Orissa rivers in India	For 16 rivers in Flood Plain
			MB = 14.0 W	Bates data of American rivers	For rivers in Flood plains
			MB = 30.8 W	Bates data of American rivers	For Incised Rivers
			ML/MB = 0.35	Jefferson data	For Incised Rivers
			$R = 37.385\ Q^{1/2}$	Jefferson data	For Incised Rivers
			ML/MB = 0.42	Jefferson data	For rivers in Flood Plains
			$R = 25.358\ Q^{1/2}$	Jefferson data	for rivers in Flood Plains
4.	Leopold et al	1964	$ML = 11.03\ W^{1.01}$	50 streams ranging from models to large rivers	
			$MB = 3.04\ W^{1.1}$		
			$ML = 4.59\ R^{0.98}$		
5.	Prus Chacinski		ML = 15.0 W	—	European Rule of Thumb

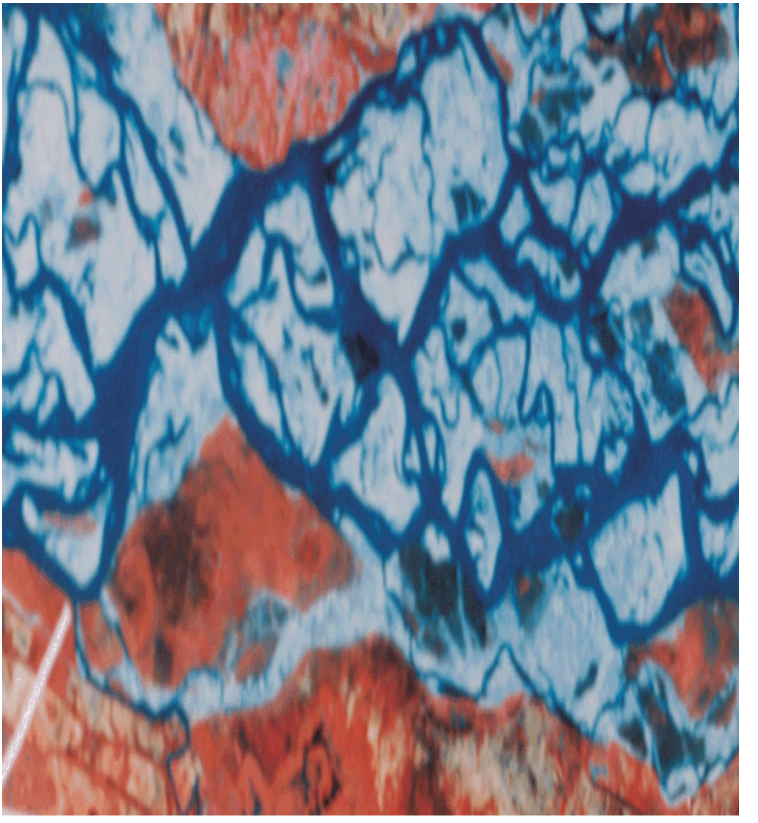
Sr.No.	Investigator	Year	Relationship	Data	Remarks
6.	Ackers & Charlton	1970	$ML = 61.19\ Q^{0.467}$ $ML = 34.11\ Q^{0.505}$ $ML/MB = 1.80\ Q^{-0.038}$	Model data	
7.	Schumm	1963 1967	$LR/LV = 3.5(W/D)^{0.27}$ $W/D = 225\ M^{-1.08}$ $ML = 193.545\ Qm^{0.34}/M^{0.74}$	Data on 47 channels	M = % of silt and clay in the perimeter Qm = mean annual discharge
8.	Chitale	1970	$LR/LV = 1.429(m/D)^{-0.077}\ S_*^{-0.052}\ (W/D)^{-0.065}$ $MB/W = 48.299(m/D)^{-0.050}\ S_*^{-0.453}\ (W/D)^{-0.471}$ $LR/LV = 1.145(MB/W)^{-0.134}$	Data of 42 rivers	$S_* = S \times 10^4$ m = size of bed material

ML = meander length in m, W = channel width in m, MB = meander belt in m,

Q = Discharge in m^3/sec, R = Radius of curvature in m, D = flow depth in m.

2.1.4 Engineering Implications of Meanders

In river engineering, meandering processes described above have vital implications. The phenomenon of progressive movement characteristic of relatively flat meanders can affect working of engineering structures like bridges, water intakes, diversion works etc. Movement of meanders results in cyclic changes in the position of main channel within the Khadir as shown in A of Fig. 2.6.



These changes are not local but are governed by changes occurring in upstream meanders. Nodal points occur in rivers where natural constraints in the form of non-erosive strata restrict downstream movement of meanders. In Ganga River, the Kasmar bluff on the north bank has thus developed a nodal point at Patna as seen in Fig. 2.7.

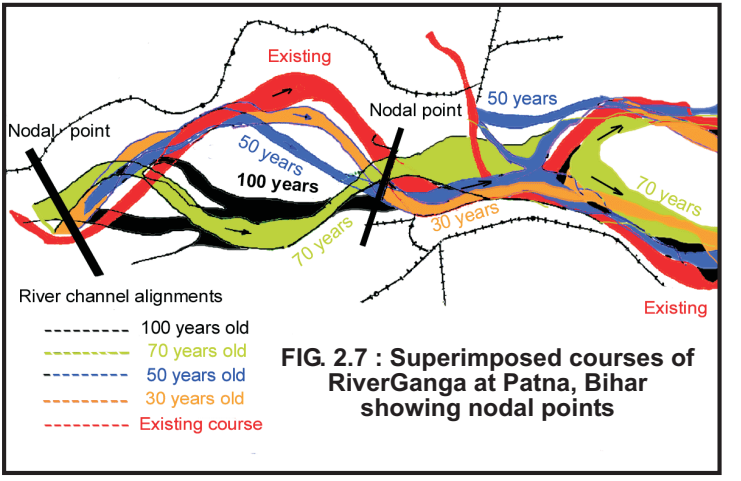


FIG. 2.7 : Superimposed courses of RiverGanga at Patna, Bihar showing nodal points

Nodal points provide good locations for constructing engineering structures since the river width here is narrower and the channel position more stable. Bridges and barrages provide artificial constraint to free movement of meanders in the downstream direction. This results in the meander loop getting squeezed and distorted immediately upstream of the structure posing threat to the safety of approach banks as indicated in B of Fig 2.6. Progressive erosion in acute bends results in development of cut offs across the neck, schematically shown in C of Fig. 2.6.

If structures are either proposed or already constructed in such rivers, it is likely, that occurrence of cut offs will affect them. In extreme cases, a cut off can render an existing structure obsolete and redundant by short-circuiting the river reach wherein the structure is sited.

2.2 BRAIDING PATTERN

Formation of braiding pattern is popularly attributed to heavy sediment load in a river having a wide and shallow cross section leading to sediment deposition during falling flood when transporting capacity is reduced quickly and appreciably. Implication of the channel section being wide and shallow is that non-uniformity in flow distribution; sediment size and sediment transport across the section is then obtained more easily. These conditions favour formation of islands and braiding pattern emerges. Brahmaputra River is a notable example of braided river as shown in Photo 2.8. Photo 2.8 shows two types of braided planforms, namely, (i) interlaced type - where number of channels intersect each other and (ii) island type - where the channels rarely cross each other. Thus, long islands are formed.

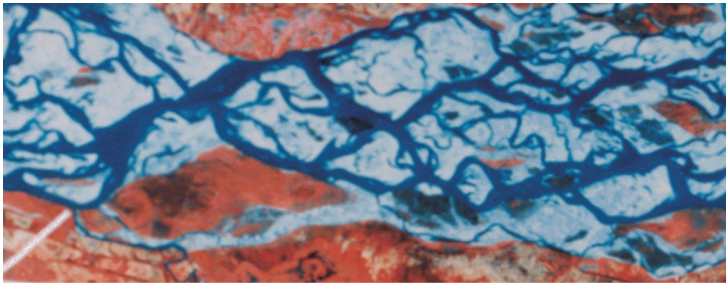
The islands in braided rivers travel downstream. Discharge carrying capacity also changes over the period causing changes in the channel alignment. Such changes cause bank erosion. Such activity is shown in Photo 2.9.

The braiding pattern when accompanied by aggradation results in lateral shifting of the river course rendering the river unstable both in plan and elevation. The Kosi in north Bihar was a well-known example of this type of river prior to construction of flood embankments. In a period of 200 years, the river shifted over 122 km from east to west as shown in Fig. 2.10.

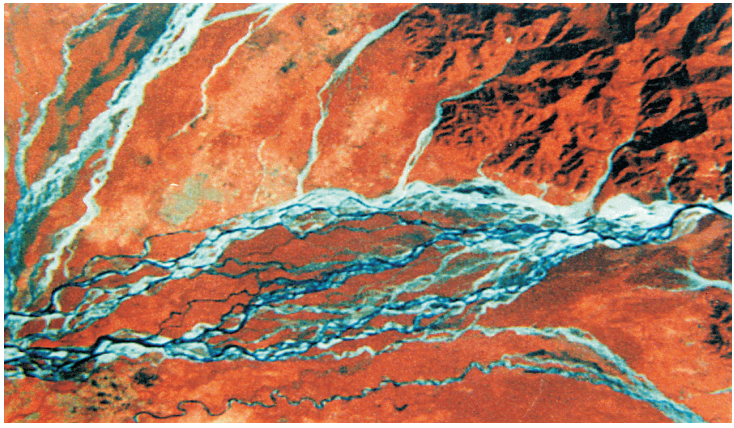
Such conditions are very unfavourable for engineering structures. The aspect of stability is, therefore, required to be examined when a project is conceived in a braided river.

Many braided streams have been trained to provide a single deep meandering course by means of lateral constriction works. Portion of the Danube River in Hungary trained in this fashion is shown in Fig. 2.11.

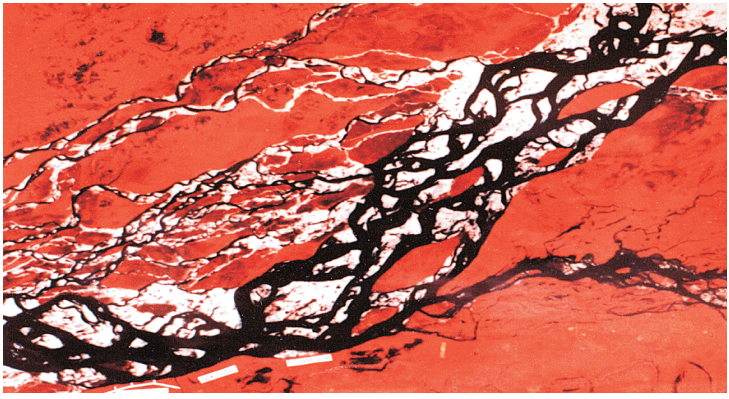
Such a single course has more capacity for conveyance of water and sediment than a braided river. This kind of river training is, however, possible when tendency for significant aggradation is absent.



Interlaced type braided channel – Brahmaputra River, Assam



Island type braided channel – Luhit River, Assam



Combined type braided channel – Brahmputra River, Assam

Photo 2.8 : Examples of braided river channels

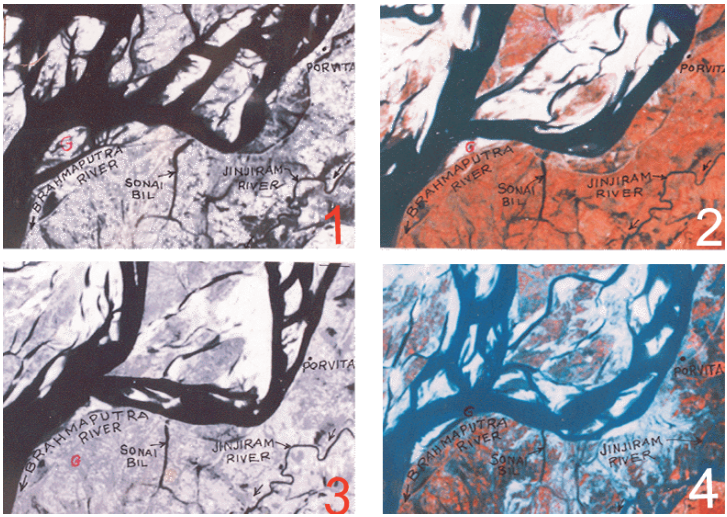


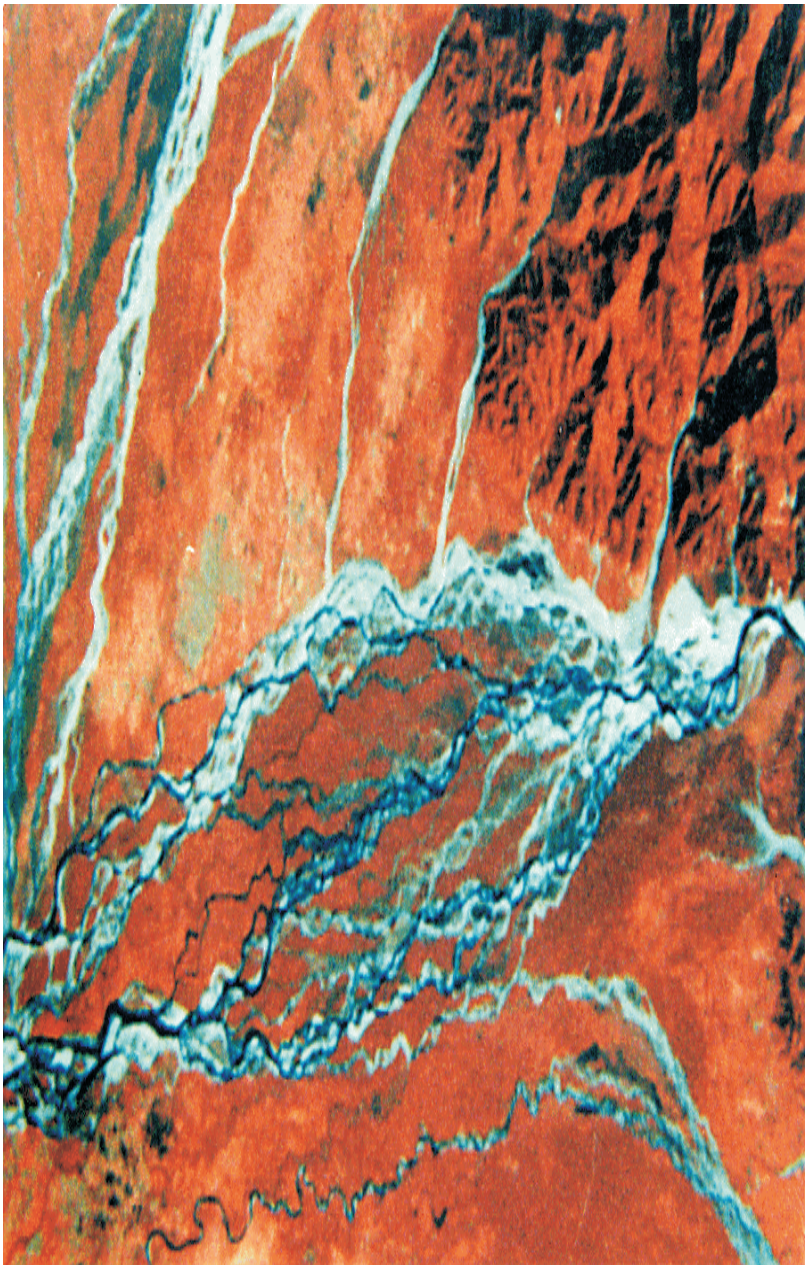
Photo 2.9 : Progressive development of channels, shifting of islands and consequent bank erosion in river Brahmaputra, Assam

2.3 BED FORMS

These are deformations of river bed caused by the flow. In a laboratory flume such deformations can be generated by progressively increasing the velocity. The sandy bed then develops bed forms in the sequence of no movement, threshold movement, ripples, dunes, transition and anti dunes schematically indicated in Fig.2.12.

Threshold movement occurs when the bed shear or bottom velocity achieves the critical value required for movement which is roughly given by the equation $T_c = 0.06\ (r_s - r)\ d_{50}$ wherein T_c is the critical shear stress in kg/cm^2, r_s is the specific weight of sediment in kg/cm^3, r is the specific weight of water in kg/cm^3 and d_{50} is the size of sediment grains in them such that 50 percent of the particles are finer than this size.

Ripples are small bed forms obtaining after threshold movement stage and have a characteristic of downstream movement. Wavelength is normally less than 0.3 m and wave



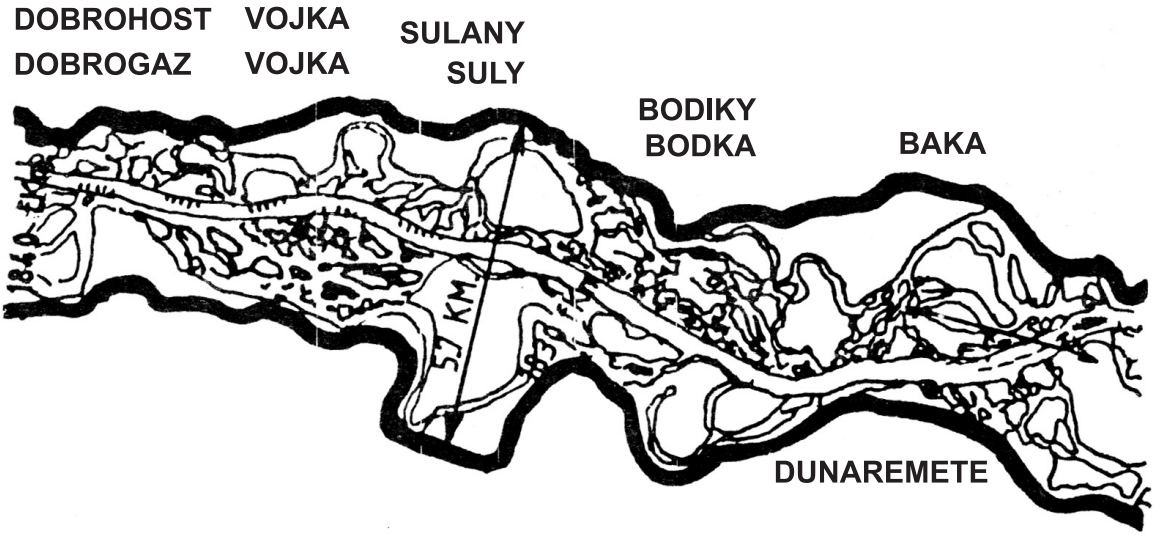


Fig.2.5 : Training of Danube river in Hungary

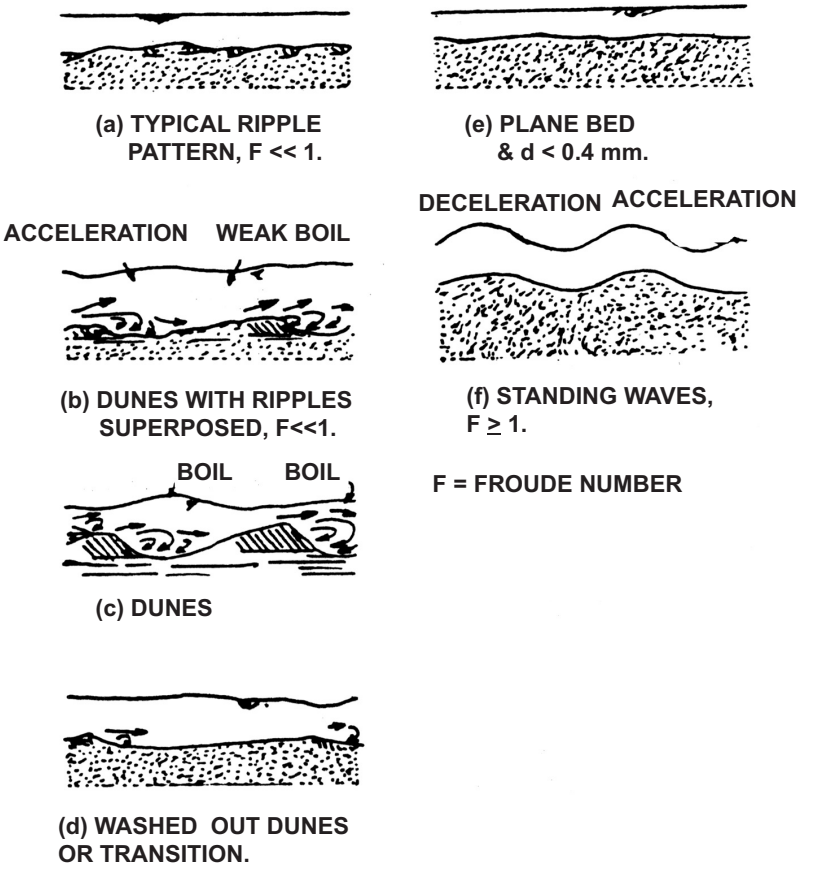


Fig.2.12 : Schematic depiction of some of the types of bed forms

height less than 3 cm. For ripple formation the size of bed material needs to be less than 0.4 to 0.6 mm. Dunes have wave length and wave height greater than ripples and these are dependent on flow characteristics. They also move downstream and are associated with sub critical flow regime (with Froude no. less than 1). As the flow approaches critical condition, when Froude Number $(V/\sqrt{gD})$ approaches 1 (where V is velocity in m/sec, D depth in m), dunes start getting washed out due to high velocity. This is the transition bed form, which is some times called flat bed. With Froude Number equal to or greater than I, antidunes are formed whose wavelength is roughly equal to $2\pi V^2/g$ where V is the velocity in m/s and g is the gravitational acceleration. Their height depends on flow characteristics. Antidunes are sand waves of sinuous shape in phase with gravity water surface waves and may move upstream, downstream or remain stationary depending on flow characteristics.

Against various bed forms described above observed in the laboratory flume, the bed forms normally met with in a river are ripples, dunes and washed out dunes. Important characteristics of bed forms having wide implications in river engineering are their magnitude and mobility. Table 2.2 gives characteristics of bed forms in Brahmaputra river in its lower reach in Bangladesh[2.6].

Sand waves of 7.55 to 15.25 m height and 183 to 915 m length were observed moving about 204 m in a day down the river. The depth of flow at this stage varied from 6.08 to 9.15 m. Similarly in the case of Mississippi river between New Orleans and Old river, a distance of 320 km, it was observed that wave heights of as much as 9.15 m were obtained in 24.4 m depth of water[2.7]. These sand waves were found to change systematically. They became larger with increase in discharge and smaller with decrease in discharge.

Implications of such bed forms are varied. Movement of big sand waves across bridge piers or intake wells may at one stage bury them in sand while at the other stage expose them excessively rendering them unsafe. Secondly, measurement of river discharge by velocity-area method often consumes a day.

Table 2.2 : Characteristics of Bed Forms in the lower Reach of Brahmaputra River

Characteristics	Small Size Ripples	Big Size Ripples	Dunes	Sand Waves
Range of Wave Height (WH)	Upto 0.3 m	0.3 to 1.5 m	1.5 to 7.6 m	7.6 to 15.2 m
Range of Wavelength (WL)	Upto 1.5 m	3.0 to 152 m	42.7 to 488 m	183 to 914 m
Range of bed form index (WH/WL)	1:5 to 1:20	1:6 to 1:100	1:30 to 1:60	1:25 to 1:100
Maximum amount of movement in 24 hours period	—	246 m	159 m	640 m
Average amount of movement in 24 hours period	3 m	122 m	67 m	204 m

Big sand waves moving across the measuring section can, therefore, affect accuracy of discharge measurement and cause scatter in the plotting of observed data in preparing stage-discharge curves, which are also termed rating curves. One more aspect is about Rugosity Coefficient in a flow formula, which depends largely on bed roughness governed by bed form. Estimation of velocity using a Manning type formula, therefore, requires proper assessment of bed form and associated value of Rugosity Coefficient.

Relationship defining type of bed form as a function of hydraulic radius R in m, slope s, mean velocity V in m/sec and grain size in mm was evolved by Simons and Richardson as shown in Fig. 2.13.

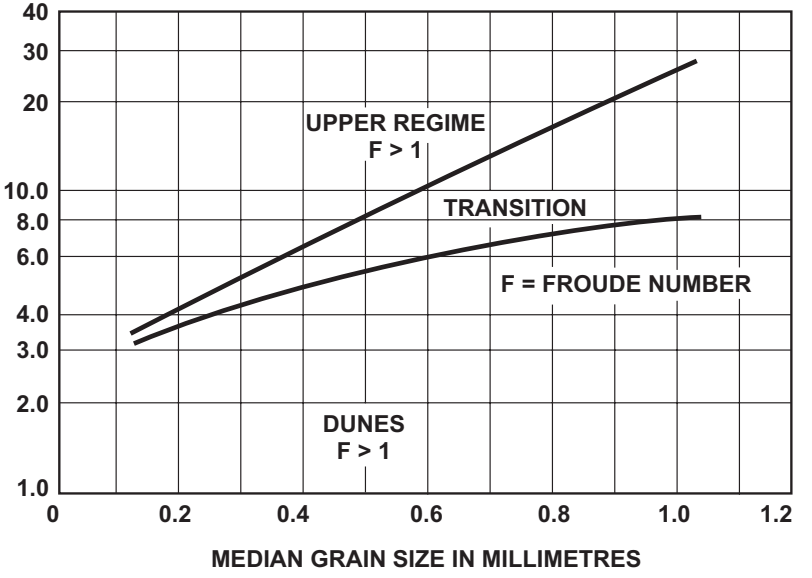


Fig. 2.13 : Relation of stream power and median grain size to form of bed roughness

Recent studies suggest that the lower regime of bed forms will occur when the ratio

$$\frac{V^4}{g^2 D^{1/2} d_{50}^{2/3}}$$

is less than 1 x 10^3, that the upper regime of bed forms will occur when the ratio is greater than 4 x 10^3 and that the bed will be in transition if ratio is in between these values. In the above ratio, V is the mean velocity in m/sec, D is the mean depth in m and d_{50} is the median grain size in mm.

2.4 RIVER FORMULAE

Schumm[2.8] dealt with interdependence of various river parameters in a qualitative way. In a river system, the parameters water discharge Q, bed material load Q_s, bankful stage width W, corresponding average depth D, slope S, meander wave length

ML, ratio of channel length to valley length P etc. all interact and change, results in changes in the remaining parameters. Using a plus exponent for increase and minus for decrease, Schumm indicated the nature of change brought about in channel morphology by change in water discharge and sediment load. The associated changes were qualitatively expressed by him in the following form.

$$Q^{+} \cong \frac{W^{+} D^{+} ML^{+}}{S^{-}} \qquad Q^{-} \cong \frac{W^{-} D^{-} ML^{-}}{S^{+}}$$

$$Q_s^{+} \cong \frac{W^{+} ML^{+} S^{+}}{D^{-} P^{-}} \qquad Q_s^{-} \cong \frac{W^{-} ML^{-} S^{-}}{D^{+} P^{+}}$$

Lane[2.10] presented a qualitative relation

$$Q_s \cdot d_{50} \cong Q.S$$

wherein d_{50} is the size of the sediment as defined earlier.

Quantitative proportionalities were developed by Langbein [2.11]. He advanced the hypothesis of uniform distribution of changes among the dependent variables and evolved structure of an exponential relationship between dependent and independent variables. By considering discharge to be an independent parameter he determined the exponents in the formulae of the type.

Width = coefficient x discharge

The proportionalities so obtained by him were

$$W \alpha Q^{0.5}$$
$$D \alpha Q^{0.33}$$
$$S \alpha Q^{-0.17}$$
$$V \alpha Q^{0.17}$$

Empirical formulae for primary variables such as width, depth and slope were evolved by Lacey, Blench and several other authors. More important of the formulae are listed in Table 2.3[2.9].

Table 2.3 : River Formulae

S.No.		Author Formulae
		Width Formulae
1.	Lacey	$W = 4.836\ Q^{0.5}$
2.	Blench	$W = F_b^{0.5}(F_s^{-0.5})Q^{0.5}$
3.	Nixon	$W = 2.988Q^{0.5}$
4.	Pettis	$W = 4.438\ Q^{0.5}$
5.	Statistical	$W = 1.434\ Q^{0.949}\ D^{-1.237}$
6.	Statistical simplified	$W = 1.6Q\ D^{-1.5}$
		Depth Formulae
7.	Lacey	$R = D = 0.473Q^{0.33}f^{-0.33}$
8.	Lacey	$R = D = 1.34q^{0.67}\ f^{-0.33}$
9.	Blench	$D = F_b^{-0.67}F_s^{0.33}Q^{0.33}$
10.	Nixon	$D = 0.539\ Q^{0.33}$
11.	Pettis	$D = 0.635\ Q^{0.30}$
12.	Statistical	$D = 1.339\ Q^{0.767}\ W^{-0.808}$
13.	Statistical Simplified	$D = 3.6\ Q^{0.8}\ W^{-1.0}$
		Slope Formulae
14.	Lacey	$S = 0.000309\ f^{-1.667} – Q^{-0.167}$
15.	Blench	$S = 0.00684\ \ F_b^{0.833}\ F_s^{0.833}\ Q^{-0.167}$
16.	Lane	$S = 0.0042\ Q^{0.25}$
17.	Statistical	$S \times 10^4 = 232\ Q^{7.200}\ W^{-7.767}\ D^{-9.600}$
18.	Statistical Simplified	$S \times 10^4 = 7.5\ M^{0.5}$
		Velocity Formulae
19.	Lacey	$V = 4500$ RS for bed material size <0.2mm
20.	Lacey	$V = 44.59\ R^{0.75}\ S^{0.50}$ for bed material size between 0.2 and 0.6 mm.
21.	Lacey	$V = 10.81\ R^{0.67}\ S^{0.33}$ for bed material size between 0.6 and 2.0 mm
22.	Lacey	$V = 6.084\ R^{0.6\ 25}\ S^{0.25}$ for bed material > 2 mm.
23.	Nixon	$V = 0.6213\ Q^{0.17}$
24.	Pettis	$V = 0.4974\ Q^{0.2}$
		Formulae for Bed Material size
25.	Statistical	$M \times 10^4 = 571\ Q^{14.31}\ W^{-15.103}\ D^{-18.379}$

Notes: (i) All formulae are in M. K. S. units except that of Blench

(ii) In Blench formulae Fs was assumed as 0.1 and F_b = 1.9 $m^{O.5}$ for size of bed material in between 0.2 and 2.00 mm.

(iii) Blench formulae were evolved on basis of canal and river data vide Reference 2.12. Lacey formulae were evolved on basis of canal data vide References 2.13 and 2.14. Nixon formulae were based on data of U.K. rivers vide Reference 2.15. Pettis formulae were obtained using data of U.S.A. rivers vide Reference 2.16. Statistical relations were worked out using data of Indian rivers vide Reference 2.17.

These empirical relationships are applicable to rivers having same characteristics as of the streams, data of which were considered in their derivation.

River formulae at Sr. Nos. 5, 12, 17 and 25 in Table 2.3 can be used to gain qualitative as well as quantitative design data in varied types of river regime problems[2.17]. In such application, the physical constraints in any given problem are important. The parameters involved are generally discharge Q, width W, depth D, slope S, bed material size M and sediment load Q_s. Not all of them may be free to change. Frequently some are changed purposely to effect the desired remodeling of the river. For instance, in order to improve navigation depths, river training is practiced by contracting the width. The ratio of original width of the river W_1 to the constricted width W_2 can take various values depending on the extent of contraction. Formulae 5, Table 2.3 then takes the form.

$$\frac{W_1}{W_2} = \left[\frac{Q_1}{Q_2}\right]^{0.95} X \left[\frac{D_1}{D_2}\right]^{-1.24}$$

Putting $W_l/W_2 = W_r$, $Q_1/Q_2 = Q_r$, $D_1/D_2 = D_r$
$S_1 / S_2 = S_r$ and $M_1 / M_2 = M_r$ and substituting $Q_r = 1$,
formulae 5, 12, 17,25 in table 2.3 give the following relationships

$$D_r = W_r^{-0.8}$$
$$S_r = W_r^{0.24}$$
$$M_r = W_r^{-0.25}$$
$$V_r = W_r^{-0.19}$$

Constriction in river width thus results in increase in river depth, flattening of slope, coarsening of riverbed material and increase in mean velocity of flow.

It is general experience that combining several channels of a stream into one increases its transporting capacity for sediment load. It is possible to assess changes in width, depth and velocity resulting from conversion of multi-channel river into a single channel one, the operative constraints being slope and bed material size remaining practically unaltered. The width and depth relations obtained under these conditions were:

$$W_r = Q_r^{1.66}$$
$$D_r = Q_r^{-0.57}$$
$$V_r = Q_r^{-0.03}$$

Similarly effect of Constructing flood embankments on depth and velocity can be evaluated. The most general case is the one where all parameters are free to change without constraint. In this case the relations obtained were the following.

$$W_r = Q_r^{0.50} \text{ (Assumed)}$$
$$D_r = Q_r^{0.36}$$
$$S_r = Q_r^{-0.27}$$
$$V_r = Q_r^{0.14}$$
$$M_r = Q_r^{0.10}$$

Thus the changes in river parameters are interdependent and when the ratio of variation in anyone of the parameters is known, the corresponding ratios for the other parameters can be derived.

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Chapter 3

REMOTE SENSING AND ITS APPLICATION FOR BRIDGE ENGINEERING

As a part of National Natural Resources Management System, India launched Remote Sensing Satellite IRS-1A, way back in the year 1984. Since then, there is long series of satellites like IRS-1B, 1-C, P4, P6, etc. The satellites have provided handy and effective tool for continuous and operational remote sensing data for the management of natural resources. Aligning the new routes, maintenance of the existing roads and railways, construction and maintenance of bridges, providing protection to the bridges, etc are integral parts of the communication system, in which, remote sensing has a significant share.

Use of remote sensing for application in river engineering is building up at a rapid pace. A bridge engineer is required to understand the hydraulic characteristics of river and the river behaviour in the vicinity of the bridge, etc before taking up the construction of a bridge. In view of safety of an existing bridge, he is required to design suitable river training measures. For many such aspects, study of satellite data of the past and present, collected through the satellites, has proved highly useful and dependable tool.

Changes in the river channel alignment and size, developments or decay of shoals, chars, islands, etc take place during floods. After studying the past data, accurate and dependable predictions of river behaviour can be made for long reaches covering upstream and downstream of the bridge.

Other information like existence of palaeo channels, low lying areas, reaches under active erosion, damages due to over bank flows, breaches in the embankments, damages to the existing hydraulic structures etc can be identified using satellite imageries.

Before discussing different aspects of remote sensing relevant to the bridge engineer, it is necessary to take an overview of the remote sensing technique.

3.1 REMOTE SENSING TECHNIQUE

Remote sensing is a science or an art of obtaining the information about an object, area or a phenomenon through analysis of data acquired by a device that is not in contact with the object, area or the phenomenon under investigation. For example, reading is an art of remote sensing where the eyes act as sensors, which respond to the light reflected from the page of written material and collect the data, which is analysed in the brain to obtain the information.

3.1.1 Satellites

Satellites used for remote sensing are of two types, viz, geostationary and polar orbiting. Geostationary stationary satellites are located at the equator at a very high altitude. Their speed and direction are so adjusted that a viewer on the earth sees them as stationary. Polar orbiting satellites rotate around the earth following a path from North to South Pole. Fig 3.1 shows the two types of satellite orbits with reference to the earth. The path and speed of polar satellites are so adjusted that they travel along the same path and at the same time at pre-determined regular interval.

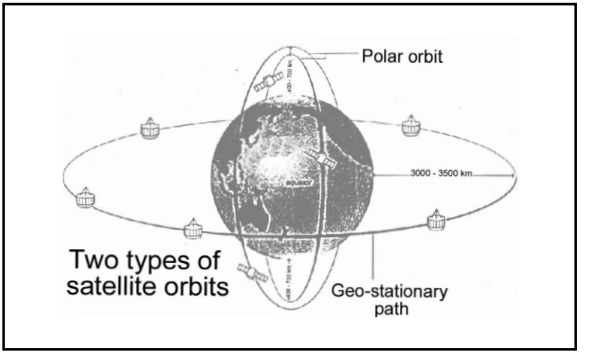


Fig 3.1 : Two types of orbits followed by Indian satellites

3.1.2 Scanning process

The difference between a photographs and imageries

needs to be understood first. A photograph covers a specific area within its view and a picture is taken covering the full area at a time. Everybody is familiar with this process. A satellite observes and records the data line by line, which is made up of many points. The point-by-point observations of the intensity of light are taken in digital form. Fig 3.2 shows the movement of satellite and the process of observation in schematic way. The process can be compared to a scanner where the source of light travels over the document, collects, and stores the data in digital form. Therefore, a satellite takes certain time to capture the data covering the specific area of interest.

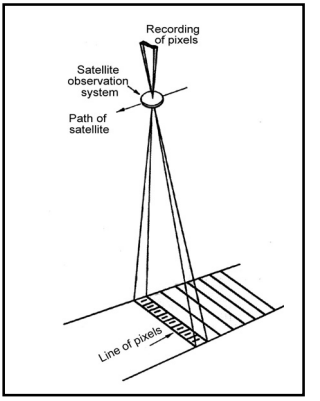


Fig 3.2 : Method of observation by satellite

The collected digital data can be converted and printed on photographic paper to form a "photograph" called in the technical term as "imagery". This process can be compared to a Xerox machine, where a document is "scanned" and the data is printed on a paper.

Aircrafts and satellites are the other most commonly used methods for data acquisition from the earth surface. Aerial photography, i.e. remote sensing of data using aircrafts, took up shape with the development of aircrafts and its suitable remote sensing cameras. Aerial photography has the advantage of flexibility of operations and higher resolution.

3.2 ADVANTAGES

Remote sensing has gained quick popularity due to the following advantages:-

i. Synoptic Coverage : Depending upon the area of interest, areas covering few hundred sq m to thousands of sq km can be suitably collected and studied. Photo 3.3 shows an example of the advantage of satellite data over index plans due to synoptic coverage. Satellite imageries of river Brahmaputra covering a length of about 150 km and about 10 km are shown on the right. Corresponding index maps prepared using other available maps are shown on the left side.

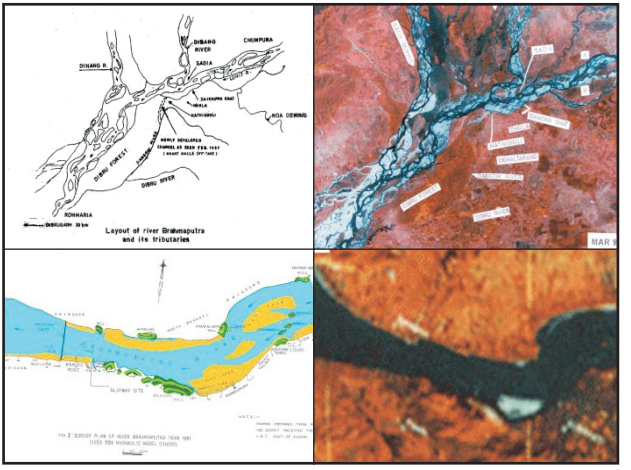


Photo 3.3 : The synoptic coverage. The right side photographs are the satellite imageries covering about 150 km (upper) and about 10 km (lower) of river length. Index maps are on the left.

Bridge engineer may consider this aspect equivalent to the plane table survey plan, drawn at a suitable scale. In this view, satellite imageries serve the same purpose, but with more accuracy and dependability.

ii. Repetitive coverage : Due to repeated passes of the satellite over the same reach, data is repeatedly acquired.

Therefore, comparisons of a dynamic phenomenon, like river channel changes, can be studied easily. Photo 3.4 shows an example of river channel alignment observed at an interval of four years. Analysis of river channel changes during the period can help to assess the changes and estimate the likely changes in future.

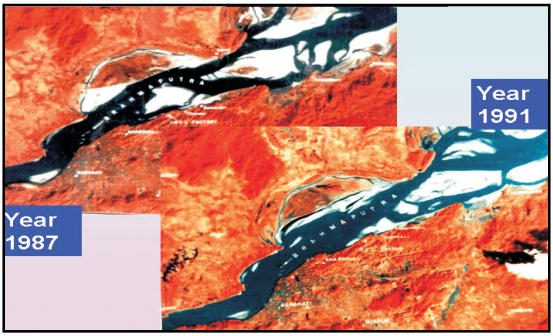


Photo 3.4 :The repetitive coverage. Imageries taken at an interval of four years show the river channel changes.

iii. Inaccessible area coverage : Marshy lands, hilly areas, etc where ground surveys could be very difficult, remote sensing technique can provide reliable and correct information. Photo 3.5 shows a river channel observed in the imageries and in the field. Difficulties in the field observation can be overcome

Photo 3.5 : Showing inaccessible river reach and corresponding satelite image.

by the use of satellite data.

iv. Amenability to computer processing : Due to the digital data collected and stored in the form of a matrix, the same becomes amenable to processing on computers. By following certain procedures, reliable identification of features and superimposition of the same to assess the changes, etc can be easily and accurately done by computer processing. Handling of large volume of data, comparison of data having different resolutions, etc becomes easier with the aid of computers.

3.3 CHARACTERISTICS OF DATA

Sensors are devices to make observations, which have sophisticated mechanism for taking observations. The detectors are designed to have specific electromagnetic properties. The detectors help to observe the desired and meaningful information from the earth's surface. Important parameters of sensors are the spatial, spectral and radiometric resolutions.

i. Spatial Resolution : Spatial Resolution is the length and width of the smallest object that can be discriminated by sensors. This is commonly known as "pixel". Smaller the size of pixel, greater is the volume of data covering the same area. Area coverage and pixel determine the scale of imagery.

The Indian satellites acquire the data in three resolutions, as below.

a. Wide Field (WiFF) sensor has a ground resolution of about 188 m and width of observation (Swath) is about 810 km.
b. Linear Imaging Self Scanning (LISS) sensor has ground resolution of about 23.5 m and swath of about 141 km.
c. Panchromatic camera (PAN) has a high resolution of about 5.8 m and swath of about 70 km.

Clarity in observation due to higher resolution, i.e. smaller pixel size can be seen in Photo 3.6. The clarity of houses, roads, trees, etc of the same reach covered by the data of different resolution is evident.

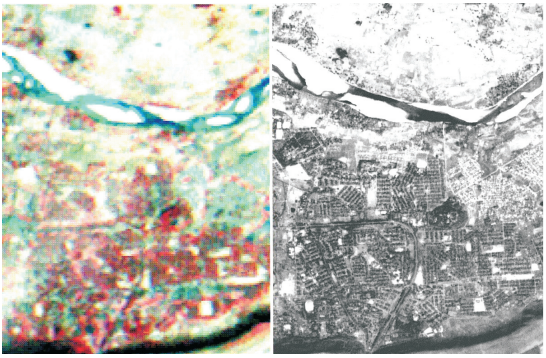


Photo 3.6 : Showing imageries of low and higher resolution covering the same area. Clarity of finer details is evident

ii. Spectral resolution : The sensitivity to different features of earth is the most important aspects considered in the spectral resolution.

The earth features can be divided broadly into three types, viz, healthy green vegetation, dry bare soil and clear lake water. It has been observed that each of these features has a clear and distinct average reflectance curves in visible electromagnetic range. However, in the nature, such features are rarely found in their pure form. The vegetation, according to their environmental conditions, have a range of reflectance depending upon the types, their water contents, stresses, diseases, etc. Similarly, the reflectance from the soil changes according to its moisture contents, soil textures, surface roughness, iron contents, organic contents, etc. The reflectance of the clear water is highly influenced by turbidity, algae contents, oil and other industrial / chemical wastes, etc.

Advantage of these characteristics is taken by taking observations in different band frequencies of electromagnetic waves within visible light range and infrared range. Different sensors are assigned separate narrow bands. Different bands help to give different identifications and classifications to the objects viewed using the data. Photo 3.7 shows rivers and adjoining land observed by different bands. Analysis of data showing the differences helps to understand the topography in a better way.

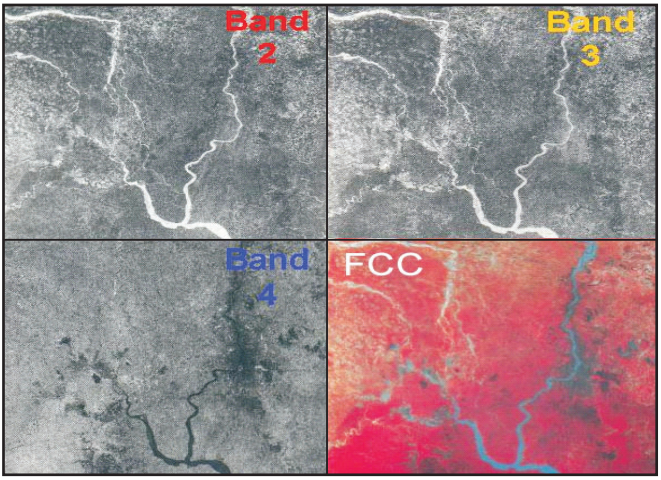


Photo 3.7 : Showing the river reach observed by different bands

iii. Radiometric resolution : It is the capability of the sensor to differentiate the reflectance from various objects on the earth. For the darkest to brightest object on the observed, one camera might show values from 0 to 32, whereas, the other from 0 to 256. This sensitivity depends up on the number of bits used to express a number in digital data.

3.4 ACQUISITION AND PROCESSING OF DATA

The steps involved before supplying the remote sensing data to the users are as follows :

a. Observations are taken using sensors like cameras, which are mounted on suitable platforms like aircrafts, satellites, etc. In case of satellites, the cameras, mounted in the satellite looking down to the earth, make continuous observations. The observations made, in digital form, are transmitted to the receiving stations on the earth. The data covering a specific reach is called as imagery of that reach.

b. The receiving stations record the observations on a suitable medium like photo papers, magnetic tapes, CDs, etc.

These also monitor the exact path, height, location, etc of the satellite when the data is received.

c. The data received from the satellite is called raw data. Distortions are possible in the observed data due to motion of the satellite relevant to earth, changes in the altitude, misalignments, position errors, curvature of earth, non-uniformity of illuminations, variations in the sensor characteristics, etc. The data, being digital, is amicable to corrections. The desired corrections are made using computers.

d. The corrected data is supplied to the user in its digital form for further analysis and interpretation using computers. The digital data is supplied on magnetic media like floppies, data cassettes, CDs, etc. The digital data requires computers having suitable softwares and some experience and expertise for further analysis and interpretation.

The supply can also be in the form of photo prints taken on photo films using computer based photo write systems. The photo print is normally black & white (B&W) when data of only one band is required. When multiple bands are to be printed, the colour of the bands is used for printing. The output prints do not show "natural" colours. For example, the green colour of the vegetation looks red in the print. Such multi-band printing is called as False Colour Composite or FCC print.

3.5 STUDY OF RIVER MORPHOLOGY

Study of river morphology gives an insight in to the changes in the rivers in planform caused due to inherent and man-made instabilities. The changes in the river planforms can be systematic / regular which can be predictable, as in case of single channel meandering flow, or complex where effect of many interactive forces result in irregular / braided plan forms. In case of braided rivers, the assessment of cause / effect relationship is slightly difficult.

3.5.1 Interpretation and analysis using remote sensing

Various characteristics of a river can be understood by

studying its planforms, their changes over periods, etc. Many of these aspects are important for a bridge engineer. River layout and planforms, like, meandering, braided, straight, etc can be seen in a synoptic view of the imagery. Further sub-division of the rivers like (a) sharp and flat, irregular and irregular, intense, type of meandering rivers (b) island type and interlaced type braiding rivers (c) straight but stable / unstable, etc can be easily identified by the use of synoptic view of the imageries.

Detailed observations and analysis of the data can help to identify many features on the ground which are relevant to the study of rivers, like hills, rock outcrops, etc; hydraulic features like old, dead and palaeo channels, oxbow lakes, dry channels, flood channels, offtake channels, large and small tributaries etc. Man-made structures like roads, railway lines, villages and towns, etc; hydraulic structures like embankments, long spurs, barrages, weirs, bridges with their appurtenant works like guide bunds, approach embankments, etc can also be identified using B&W and FCC prints in combination or separately.

Temporal data covering the desired period can help to study the configuration of river channels and other hydraulic structures in the past. Analysis of the data helps to study the shifting of river channels, development of shoals and islands, changes in size of dry weather channels, bank line changes, nodal points in meandering rivers, stable and unstable reaches, Khadir limits, etc. The study in turn helps to decide the location for abutments, waterway required from stability point of view, vulnerable reaches which may require river training / anti-erosion measures for safety of the bridges, etc.

When FCC imageries are used, a bridge engineer can visually see the changes in the colours, variation in the colour patterns in specific reaches, etc. Inspection of site helps to verify the features which can help to assess the flood prone areas, areas where shallow and stagnant water can be met with, density of forest in the bridge approaches, etc.

Once the features as observed on the ground and corresponding features observed on the photo prints of satellite imageries is understood, a bridge engineer can undertake the preliminary analysis and decide many aspects of the bridge param-

eters qualitatively. However, for reliable quantification is required, computer analysis is recommended, wherein suitable softwares and expertise in the digital analysis is necessary.

3.6 PROCUREMENT OF DATA

National Remote Sensing Agency (NRSA), of the Department of Space (DOS) is located at Balanagar, Hyderabad - 500 037. NRSA is nodal agency for the distribution of the data acquired by the Indian remote sensing satellites. NRSA has a large data bank collected by Indian satellites since 1984 till date. Data older than that is also available from the American satellite Landsat. NRSA also has a large collection of data acquired by other foreign satellites like NOAA, SPOT, etc. These data are stored and supplied to the users according to the availability. Except few areas critical from security point of view, all the data available with NRSA is fully open for any user.

NRSA requires a requisition indicating the exact requirement of the imageries. A bridge engineer newly intending to use the data may face difficulties in the procurement. Following steps are suggested which can be followed for smooth procurement of the correct data.

Based on the response of the users, NRSA modifies the methods and adds new facilities in their procedures. The most common procedure, which can be easily adopted by the new user, has been discussed below.

3.6.1 Steps for the procurement

a. Initially, an introductory letter may be sent addressed to The Head, NRSA Data Centre, National Remote Sensing Agency, Balanagar, Hyderabad – 500 037. The letter should clearly indicate purpose of studies, type of studies involved, area of interest; expected resolution; period of data to be covered for each imagery, scale of the photo print required, etc. Requisition form may also be requested in the introductory letter. Details of these items are discussed separately.

b. Based on the information, NRSA will browse the available data and issue “Proforma Invoice“ indicating suitable satel-

lites, dates of pass showing cloud free data, and cost of each photo print. More than one date of pass might be suggested within each of the period interest. In such case, the bridge engineer should make selection of suitable date of pass.

c. Bridge engineer should duly fill the requisition form and a Demand Draft covering 100% cost of the imageries should be drawn and enclosed with the requisition form and sent to NRSA. If more than one date is available in the desired period, alternate dates of pass, in order of priority, may also be indicated in the covering letter.

d. The desired data is normally made available within a period of 3 to 4 weeks. Some times, NRSA faces problems in retrieving old data, percentage of cloud cover for the desired area, missing lines of data, etc. Under such circumstances, NRSA will make use of the alternate dates specified in the covering letter. If such alternate dates are not available, NRSA will come back to the user for suggesting new period for the date of pass. Fresh look at the suitable date of pass may have to be taken under such conditions.

3.6.2 Details for the requisition

Explanation regarding the details to be filled in the requisition form is given below for ready reference.

i. Period of Data

Normally, for the morphological studies of a river, dry weather data is required. Morphological changes in a river take place due to floods during monsoon season. Changes in the river planforms, shifting of channels hardly take place during lean period. Therefore, suitable data of lean period, i.e. from October to December and January to May is always acceptable. The exact period should be specified for clarity, viz, October 1986 to May 1987; October 1991 to May 1992; October 1996 to May 1997, etc.

For full clarity, only cloud free data should be requisitioned. Normally, sufficient choices for the date of pass are available in a specified year.

ii. Area of interest

The user (i.e. the bridge engineer) is required to decide and specify the area of interest. The area is indicated as a rectangle of sides parallel to north-south and east-west direction. This can be specified as follows

a. Longitude and latitude of the lower left corner and upper right corner of the rectangle

b. Survey of India map sheet Number can be indicated. This is normally readily available with the bridge engineer. Therefore, this can be the most common method to indicate the desired area.

c. Alternatively, the longitude and latitude of the bridge can be indicated. NRSA can treat this location as centre point of the map sheet to be supplied. This facility is applicable for the high resolution data if IRS 1C and 1D, P4, etc. These data are available from the year 1996.

d. Path and row number of the desired satellite can be indicated. NRSA has prepared and published maps showing path and row numbers of different satellites. For a new user, this may not be easily available. Experienced user and the users who process the data on computers are normally familiar with this alternative.

iii. Reach to be covered

River reaches to be studied with the help of satellite imageries upstream and downstream of the bridge depends up on the likely effect extending upstream and downstream. Therefore, the lengths can be different depending up on the type of rivers, its horizontal and vertical stability, etc. In case of meandering and stable rivers, the length can be three meanders upstream and two meanders downstream of the bridge. For braided rivers, the length can be four to five times its maximum width in the upstream direction and three times the maximum width downstream of the bridge.

From the point of view of safety of the bridge, if the analy-

sis indicate necessity of protection / training measures in the upstream, then the length of study reach can be accordingly extended in the upstream direction.

iv. Bands and resolution

Normally, the LISS data is found sufficient for the studies. However, for important structures at critical locations and structures located in the urban reach, additional data with higher resolution i.e. say PAN data can be procured covering the desired river reach.

v. Number of photo prints

Bridge engineers would normally require data on photo prints of bands 2, 3 and 4 for FCC prints, or of only band 4 for B&W prints. For conducting preliminary studies, FCC data of the latest available period is required, whereas, for conducting studies in detail, data at an interval of 4-5 years till the latest data is normally sufficient. Procuring the FCC data for the oldest and latest imageries and B&W of band 4 for the other years is normally sufficient for analysis of river behaviour. In addition to the above, as discussed above, PAN data covering a specific reach can be procured.

3.7 CONCLUDING REMARKS

The above discussions regarding remote sensing give only introductory information. An attempt to procure the data and / or a visit to NRSA, Hyderabad will definitely help to understand the procurement and analysis of the information in a better way.

The different parameters for the bridges, which can be studied with the help of satellite data, are discussed at their proper location in the subsequent chapters.

REFERENCES

3.1 “Remote Sensing and image interpretation”, T M lillesand, R W Kiefer, Published by John Wiley & Sons Inc, New York, Singapore, 1994.

3.2 Lecture notes on “Application of Remote sensing and GIS techniques in water resources development and management “ Edited by National Water Academy Khadakwasla, Pune – 411 024.

3.3 “Satellite data products from NRSA Data Centre”, information brochures and booklets published by NRSA, Hyderabad from time to time.

■ ■ ■

Chapter 4

PRINCIPLES OF RIVER TRAINING AND PROTECTION WORKS

Determination of design discharge for a bridge is some times required to be based on flood marks left during unprecedented abnormal floods. A flow formula is then used to estimate discharge corresponding to flood marks. The river section is often composite comprising flow section with one or more deep channels and an overbank area carrying spill flow. Allowance for such variation is required to be made in application of the flow formula, suitable procedure for which needs to be known.

Excessive constriction of waterway by approach banks results in rise of water level which is termed afflux. Apart from construction of waterway, afflux is also caused due to pier obstruction. The extent of afflux is an important consideration in fixing bridge waterway.

In the case of alluvial rivers, the normal Indian practice is to provide constricted waterway for a bridge when river width is more than the width given by the Lacey formula. In this case increase in discharge intensity at the bridge section, causes bed scour on account of sediment transporting capacity becoming locally more than sediment supply. Estimation of scour and its effect on afflux in such rivers are also important design aspects.

Width and depth of alluvial rivers are primarily governed by discharge. Relationships have been evolved for estimation of normal dimensions of these parameters. Width and depth formulae evolved by Lacey are commonly adopted in fixing bridge waterway and estimating scour depths. Limitations of such formulae and their implications, however, need to be appreciated to enable judicious application.

Thus certain topics in open channel flow hydraulics are of special relevance to bridge engineering which are discussed below. These comprise velocity distribution and discharge formulae, afflux due to various reasons and consequent scour, and formulae for river parameters like width and depth.

4.1 VELOCITY DISTRIBUTION AND FLOW FORMULAE

Variation in velocity along a vertical in channel cross section is known to follow logarithmic distribution, maximum velocity being at the top and minimum at the bottom. Mean velocity is obtained at 0.632 depth from top. Average of velocities measured at 0.2 and 0.8 depths from top also gives the mean velocity. The velocity distribution along a vertical depends on depth and relative roughness of bed material. If bottom roughness is increased, the bottom velocity is decreased and top velocity correspondingly increased. Distribution curves for different bottom roughnesses are shown in Fig. 4.1.

River discharge can be determined as A x V where A is area of flow section and V the mean flow velocity. Sectional area can be derived knowing cross sectional dimensions. Mean sectional velocity of flow in a river can be estimated by adopting any of the following formulae.

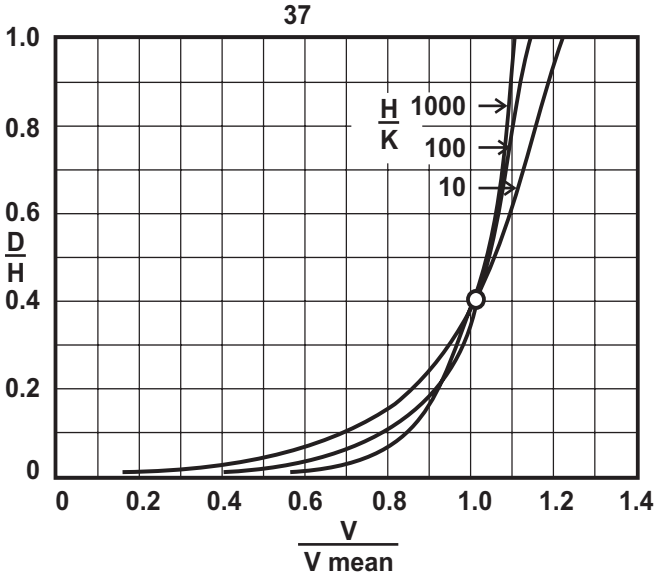
(i) $$V = \frac{R^{2/3} S^{1/2}}{n}$$ Manning Formula

(ii) $$V = CR^{1/2} S^{1/2}$$ Chezy Formula

(iii) $$h = \frac{fLV^2}{8Rg}$$ Darey Weisbach Formula

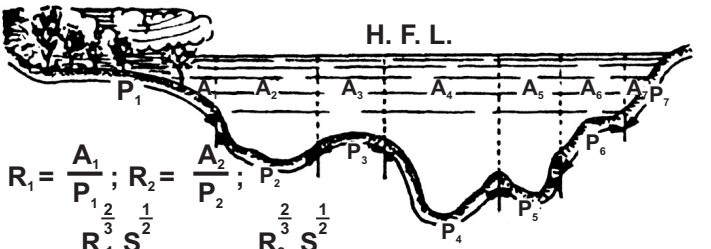
(iv) $$\frac{V}{V*} = 6.25 + 5.75 \log \frac{R}{K_s}$$ Logarithmic formula

wherein V is the mean velocity in m/s of the cross section, R is the hydraulic mean depth in m, S is the energy slope which becomes equal to water surface slope and bed slope under uniform flow conditions, n the rugosity coefficient, C the Chezy coefficient, h is the head loss in m, f is the friction factor, L is the length in m over which head loss h is obtained, g is gravitational acceleration, V* is friction velocity in m/s equal to $(To/\rho)^{1/2}$ or $(g\ RS)^{1/2}$ and K_s is the bottom roughness in m. All the above formulae are of the same form but having different structure and can be rearranged and expressed as:



D = TOTAL DEPTH OF HEIGHT.
K = BOTTOM ROUGHNESS.
H = HEIGHT FROM BOTTOM ON A VERTICAL ll
V = VELOCITY AT HEIGHT H.
V_{mean} = MEAN VELOCITY OVER A VERTICAL

Fig. 4.1 : Vertical velocity distribution with different bottom roughnesses



$$R_1 = \frac{A_1}{P_1}\ ;\ R_2 = \frac{A_2}{P_2}\ ;$$

$$V_1 = \frac{R_1^{\frac{2}{3}}\ S^{\frac{1}{2}}}{n_1}\ ;\ V_2 = \frac{R_2^{\frac{2}{3}}\ S^{\frac{1}{2}}}{n_2}$$

$Q_1 = A_1 V1$ ... $Q_2 = A_2 V_2$

$Q = \Sigma Q_n$

Fig. 3.2 : Discharge computation for a river with composite cross section

$$\frac{V}{V_*} = \frac{R^{1/6}}{n} = \frac{C}{g^{1/2}} = \left[\frac{8}{f}\right]^{1/2}$$

wherein n, C and f are measures of boundary roughness in Manning, Chezy and Darey Weisbach formulae.

In case of Manning formula, value of rugosity coefficient n is given by $n=K_s^{1/6} / 76.5$ wherein K_s is median grain roughness diameter d_{50} in mm such that 50 percent of the material is finer than this size. Data of values of n for different sized bed material and in different types of channels is given in Tables 10.2 and 10.3 of Chapter 10.

River cross section is rarely uniform. It may have one or more deep channels and also overbank spill area. There can be considerable difference in n values in channel portion and overbank area. In such a composite cross section, discharge of each portion of the section is required to be worked out separately and then added to arrive at total discharge as indicated in Fig. 4.2.

4.2 AFFLUX IN RIVERS WITH NON SCOURABLE BED

Afflux is defined as rise of flood level of the river upstream of a bridge as a result of the obstruction to natural flow caused by the construction of the bridge. It is caused either due to constriction of waterway by approach banks or on account of obstruction to flow caused by piers. Afflux results in increase of velocities in constricted section and at the obstructions. When riverbed has nonscourable strata or material, the increased velocities cannot scour the bed and therefore the boundary remains unchanged. Estimation of afflux under this condition is relatively simpler. When, however, the riverbed is sandy, it can scour easily due to increased velocities at the bridge. Estimation of afflux and scour in this case involves unstable boundary condition and is, therefore, more involved. Condition of non-scourable boundary is first considered below.

4.2.1 Afflux due to constriction of waterway

Specific energy E_f in m per m^3 of flow is the energy measured with respect to bottom and hence comprising potential

head D and velocity head $V^2/2g$ also in m. Thus E_f equals summation of D and $V^2/2g$. When bridge waterway is constricted by making the bridge narrower than the channel width, discharge intensity q increases at the bridge section. Change in q brings about corresponding changes in D and V but not in E_f. The interrelationship amongst these parameters is depicted graphically in Fig. 4.3 reproduced from Reference 4.1.

Following a line of constant specific head E_f in the subcritical flow region, it is seen that river discharge with a particular specific head can pass through the bridge with different discharge intensities q implying different constrictions. Bigger the constriction, higher the value of q and lesser the depth D. The velocity head $v^2/2g$ correspondingly becomes higher. Such progressive constriction of bridge waterway and increase in discharge intensity is, however, possible only up to a limit which is reached at critical stage. If further constriction is caused, the available specific energy will be insufficient to pass the discharge and hence flow will head up on upstream side of the bridge to force the discharge through. This heading up then becomes apparent as afflux and its effect is felt as backwater over long reaches. On the down stream side the energy built up in formation of afflux is dissipated by generation of hydraulic jump. Conditions leading to afflux on upstream and jump on the downstream side of constricted waterway of a bridge are possible when the bed is nonscourable. When riverbed is sandy, increased velocities in the constriction develop bed scour. This condition is considered separately.

In rivers with clayey bed, sufficient bed scour to obviate afflux may not be formed initially. If constriction is excessive, significant afflux can result leading to formation of a hydraulic jump. Under such conditions, foundations of piers and abutments are required to be designed sufficiently deep to ensure safety. The safety criterion is to assume that full scour may be obtained eventually. Full scour required for obviating jump formation is such which will avoid development of any significant afflux and consequent supercritical flow in the constricted bridge section.

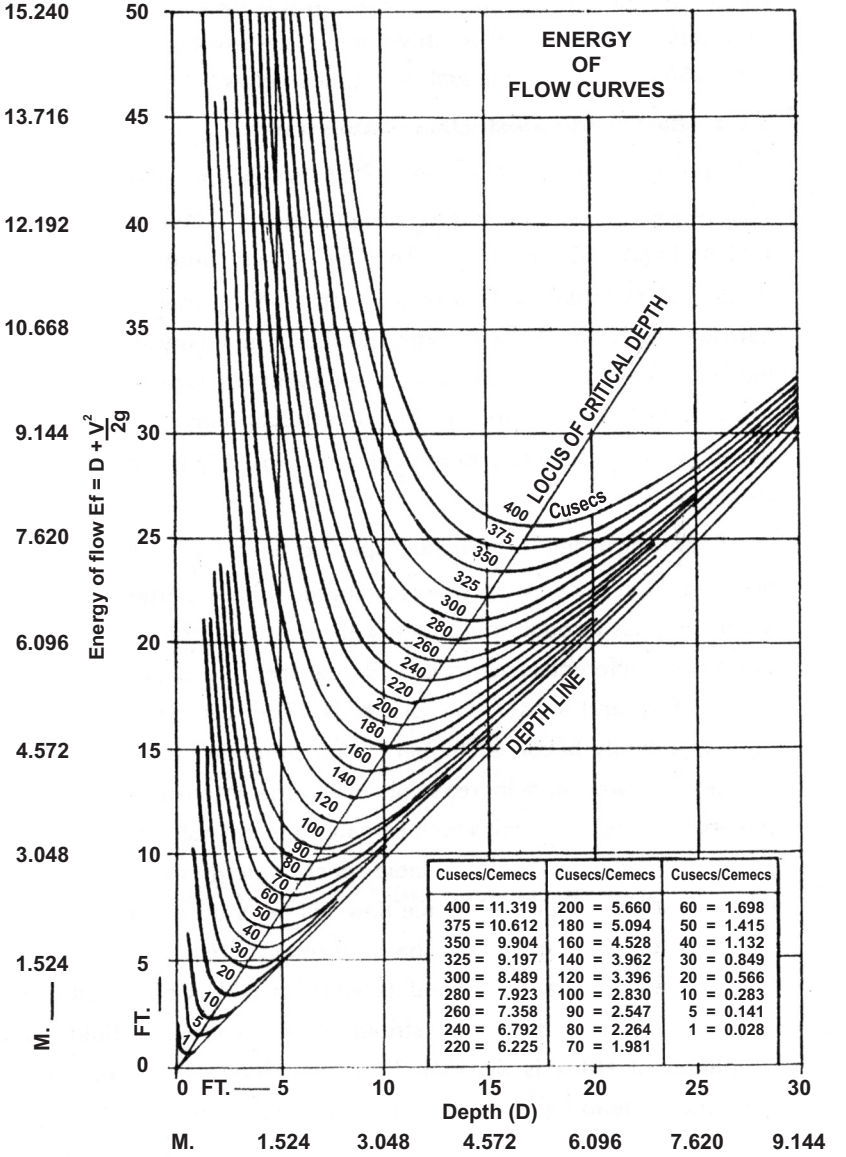


Fig. 4.3 : Relationship between specific energy, depth of flow and discharge intensity

4.2.2 Afflux on account of obstruction to flow

Even if the river constriction on account of approach banks is not excessive and bridge waterway is wider than that causing critical discharge intensity, afflux can still be generated due to obstruction to flow by piers. Separation eddies forming at such obstructions result in loss of head which shows as afflux upstream of the bridge. One of the formulae in popular use in India for estimation of afflux at bridges is due to Molesworth[4.8] given below.

$$h = [(V^2 / 17.88) + 0.01524] [(A/a)^2 - 1]$$

wherein h is afflux in m, V is velocity in m/s in unobstructed stream, A is unobstructed sectional area of the river in m^2 and a is obstructed sectional area of the river in m^2. Other formulae in wide use are due to Negler and Yarnell[4.3][4.4]. More recently, the Bureau of Public Roads U.S.A., have adopted procedure for estimation of afflux detailed in Hydraulics of Bridge Waterway[4.5].

Affluxed waterway has relevance in providing free board and clearance, norms for which are given in paras 4.4 and 4.5.

4.3 AFFLUX AND SCOUR IN ALLUVIAL RIVERS

In case of alluvial rivers, the river bed comprises scourable material like sand. Constriction of bridge waterway causes increase in discharge intensity and increase in velocity which result in bed scour. Scour in riverbed increases the depth of flow and reduces velocity and hence the scouring capacity. When excess of sediment transporting capacity in the constriction beyond normal capacity in unconstricted reach is neutralised by progressive bed scour during rising flood of a hydrograph, no further scour occurs. This depth with full scour approximates normal scour depth in the same river adjusted for the increased discharge intensity on account of constriction. With development of full scour, afflux becomes negligible as witnessed in alluvial rivers carrying sustained floods. In case of Ganga river at Mokamah bridge, the afflux observed was only 5 cms for a flood discharge of 51000m^3/s[4.6].

When the river carries a flashy flood, time may be inadequate to develop full scour, even if the river is alluvial and bed is scourable. Substantial afflux can then occur. Scour depth and afflux for a flashy flood can be estimated by making computations for progressive rise in flood discharge starting with the lowest stage. Flood hydrograph is converted to a step diagram, each step pertaining to a small rise in discharge obtained for a specific duration. Smaller this specific duration, higher will be the accuracy. For the first rise in discharge, afflux is initially computed for no scour condition. Scour depth for increased velocity is then estimated using any of the appropriate sediment transport functions. Numerous such functions have been given in Reference 4.7. Time available for scour is the time during which, that particular discharge stage is experienced. This time is woven in the method of estimation of the scour depth. With this scour depth, the afflux is recomputed. By iteration process the afflux and scour for the first rise in flood discharge is finalised. Next rise in discharge is then considered. Entire flood hydrograph is thus covered up to the peak point. Afflux and scour values associated with the peak stage of a given design hydrograph provide necessary design data. More detailed procedure is explained in Reference 4.8.

4.4 FREE BOARD

The free board above designed flood level in case of rivers and full supply level in case of canals, including the afflux, to the formation level of the railway embankment or guide bund should not be less than 1 m. In case where heavy wave action is expected, the free board should be further increased to allow for the same.

Vertical wave run up depends on bank slope, wave height and wave period. Wave characteristics such as wave height, length and period in turn are dependent on wave velocity, straight length over which it is incident, termed fetch, wind velocity and depth of water. Knowing this design data, it is possible to estimate wave run up. Stevenson formula modified by Moliter is in popular use for estimation of wave height which is given below.

$$h = 0.0322\,(V.F)^{1/2} + 0.76 - 0.89\,F^{1/4}$$

wherein h is wave height in m, F is fetch in km and V is wind velocity in km per hour.

4.5 CLEARANCE

As applied to the bridge super-structure over water channels, this is vertical height between the designed flood level (including afflux) of the stream and a point on the bridge super-structure where the clearance is required to be measured.

In the case of an arch bridge, the clearance will be the vertical height between the design flood level (including afflux) and any point on the soffit of the arch where clearance is required to be measured.

In the case of road-over or under bridge, it is the shortest distance between the moving dimensions for railway or road vehicles and the superstructure.

In case of bridges with rectangular openings, the minimum clearances normally provided are as in Table 4.1.

Table 4.1

Discharge (Cumecs)	Vertical Clearance (mm)	Limits upto which relaxable in special circumstances (mm)
Less than 3	600	300
3 – 30	600	300 – 400 (Prorata)
31 – 300	600 –1200	400 – 1200 (Prorata)
301 – 3000	1500	No relaxation is permissible
Above 3000	1800	No relaxation is permissible

In case of arch bridges, minimum clearances measured to the crow of the arch are indicated in Table 4.2.

Table 4.2

Span of Arch (m)	Clearance from crown of arch
Less than 4	Rise or 1200 mm whichever is more
4 – 7	2/3 Rise or 1500 mm whichever is more
7 – 20	2/3 Rise or 1800 mm whichever is more
Above 20	2/3 Rise

In aggrading rivers where the bed has a tendency to rise the clearances provided should be suitably augmented. Bigger clearance may be required when river navigation is to be catered for. Possibility of floating trees being brought down from upper catchment needs to be considered and adequacy of clearance checked on this accord.

4.6 LACEY FORMULAE

Lacey originally evolved width and depth formulae for stable alluvial canals. These were the following. [4.9]

$$P = 4.836\ Q^{1/2}$$

$$R = 0.473 \left(\frac{Q}{f}\right)^{1/3}$$

wherein P is wetted perimeter in m, Q is discharge in m^3/s, R is hydraulic mean radius in m and f is the silt factor which is given as 1.76 $m^{1/2}$, m being weighted mean diameter of bed material in mm. Working out f value from frequency diagram" of river bed material is indicated under para 6.8.1 (i) of chapter 6.

On the basis of data of a few rivers, Lacey showed that his P:Q relationship is valid not only for canals but also for alluvial rivers. Since river sections are much wider than canals, P and R can be replaced by W and D, namely width and depth respectively.

Bridge waterway is normally designed using Lacey relationship $P = 1.811\ CQ^{1/2}$ allowing some deviation in the value of C from 2.5 to 3.5 according to local conditions. The depth of relationship $D = 0.473\ (Q/f)^{1/3}$ is employed for calculation of scour depth at bridge piers or along guide bunds adopting multiplying scour factors given in I.R. Substructure Code as given in Table 4.3.

Table 4.3

S.No.	Nature of the river	Depth of Scour
1	In a straight reach	1.25 D
2	At a moderate bend condition e.g. along apron of guide bund	1.5 D
3	At a severe bend	1.75 D
4	At a right angle bend or at nose of piers	2.0 D
5	In severe swirls e.g. against mole head of guide bund	2.5 to 2.75 D

An important aspect of the two basic Lacey formulae is that they are derived and applicable for near bankful stage along the river but are invalid and should not be used to find variation in either W or D at a place at different discharges or flood stages.

Secondly, the width formula was shown by Lacey to be applicable to rivers on basis of only 7 observations. Rivers of braided pattern are, however; known to be wider than Lacey width. For instance Brahmaputra and Sone rivers have ratios of actual width to Lacey width of as much as 12.41 and 7.8 times respectively. On the other hand in case of incised rivers the actual width can be shorter than Lacey width. In Tapi river, the width is 0.44 times that given by Lacey formula. Generally the Lacey width formula gives a better fit for alluvial rivers in flood plains. A few typical examples of variation in width ratio in different types of rivers are given in Table 4.4.

Table 4.4

Comparison of actual widths of rivers with those given by the Lacey Formula

S. No.	River	Bankful discharge m^3/S	Actual River Width (m)	Lacey Width (m)	Actual 'W'/Lacey 'W'	Remarks
1	Jhelum, J&K, India	394	103	97	1.06	Flood Plain Meandering
2	Bhagirathi, West Bengal India	1670	218	198	1.10	-do-
3	Ramganga U.P. India	6600	345	394	0.88	-do
4	Indus of Hajipur Pakistan	7050	985	1020	0.97	–do
5	Mississippi at Vieksberg, USA	42400	1380	1000	1.38	–do-
6	Tapi, Gujarat India	17000	274	630	0.44	Incised
7	Kosi, Bihar India	7050	6150	406	15.06	Flood Plain Braided
8	Brahmaputra Assam India	24700	9450	760	12.41	Flood Plain Braided
9	Sone, Bihar India	14100	4500	575	7.79	Flood Plain Braided

This limitation of Lacey formula needs to be borne in mind while using it for estimation of bridge waterways.

Deviation in respect of depth parameter is less than that of width. Better fit to field data is possible if Lacey depth formula expressed in terms of 'q' the discharge intensity in m^3/s per m width instead of 'Q' the total sectional discharge in m^3/s, giving depth 'D' in m as

$$D = 1.34\ (q^2/f)^{1/3}$$

'f' being the Lacey silt factor. This formula can be reduced to

$$D = 1.12\ Q^{0.67}\ W^{-0.67}\ m^{-0.17}$$

where 'W' is water surface width in m and 'm' the size of bed material in mm. The semi-theoretical formula derived by Laursen and Latishenkov curves in vogue in the U.S.S.R. have the same form of equation as the Lacey D,q, f formula.

The formula D,Q,f and D, q, f become identical when width and average depth of a river section are equal to those given by Lacey Formulae. When width, however, is different from that given by Lacey formula, D, q, f formula become applicable in preference to D, Q, f formula.

Important Conversions

In F.P.S.	In Mks
$n = \frac{K_S^{1/6}}{29.3}$ (K_S in ft)	$n = \frac{K_S^{1/6}}{76.5}$ (K_S in mm)
$h = \left[\frac{V^2}{58.6} + 0.05\right] X \left[\left(\frac{A}{a}\right)^2 - 1\right]$	$h = \left[\frac{V^2}{17.88} + 0.01524\right] X \left[\left(\frac{A}{a}\right)^2 - 1\right]$
$h = 0.17\ (V\ .\ F)^{1/2} + 2.5 - (F)^{1/4}$	$h = 0.0322\ (V\ .\ F)^{1/2} + 0.76 - 89(F)^{1/4}$
$P = 2.67\ Q^{1/2}$	$P = 4.836\ Q^{1/2}$
$R = 0.473\ (Q/f)^{1/3}$	$R = 0.473\ (Q/f)^{1/3}$
$D = 0.9(q^2/f)^{1/3}$	$D = 1.34(q^2/f)^{1/3}$

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Chapter 5

TYPES OF TRAINING AND PROTECTION WORKS

5.1 BRIDGE LOCATION AND ALIGNMENT

A bridge is normally located where the river section has minimum width so that the structure becomes shorter and economical. As far as possible it is aligned normal to the river. If satisfying these requirements involves a long detour, some compromise is made in siting a crossing. Care is also taken to avoid, if possible, sites where frequent changes occur in river course, tendency for aggradation or degradation is manifest and problems of bank erosion or difficult foundation conditions are required to be faced.

Approach banks in case of constricted bridges are generally aligned in line with the bridge axis and normal to the guide bunds. Approaches deviating towards downstream direction add to their safety from river loops approaching dangerously close while deviation in upstream direction exposes them more to river attack.

Location of the bridge with respect to Khadir width also determines the length and position of approach bank exposed to river attack. When bridge is constructed near one of the ends of the Khadir width and when the khadir edge consists of stiff material resisting erosion, cost of one guide bund can be saved. Longer length of the approach at other end however gets exposed and becomes vulnerable to river attack in this case. On the other hand central location of the bridge reduces length of approach open to river attack, minimises possibility of extreme obliquity of approach of the river towards the bridge but requires provision of guide bunds on both the flanks.

In general, bridge location and alignment are fixed to ensure normal approach of the river, equitable distribution of flow across bridge section and minimum possibility of river attack on approach banks.

5.2 BRIDGE WATERWAY

Larger bridge opening involves more cost. It permits haphazard sediment deposition and formation of islands. Obliquity and concentration of flood flow can then develop any where on bridge section with consequent deep scour and high velocity. In India, the practice evolved in case of alluvial rivers is, therefore, to constrict the river waterway at bridges. Thus the main advantages of constricted waterway are the resulting economy and better hydraulic performance of the bridge. The first bridge to be constricted was on Chenab river at Shershah in 1888, [5.1] now in Pakistan, shown in Fig. 5.1.

Benefits accruing from constriction were so convincing that constriction of bridges on alluvial rivers has since become a standard practice.

Extent of permissible constriction was determined by Spring and Gales[5.2] on basis of river data of maximum flood depths and velocities. Lacey on the other hand verified the validity of his relationship between wetted perimeter P and discharge Q originally evolved for alluvial canals for application to rivers. [5.3]. Parameter P can be replaced by width W in case of wide rivers giving

$$W = 4.836\ Q^{1/2}$$

wherein W is in m and Q in m^3/s. His finding was that this formula fits the river data well and hence can be adopted for determining constricted waterway of a bridge. Subsequently Sethi [5.4] and Khosla Committee[5.5] accepted Lacey formula as basis for design of waterway with suitable deviations. The current practice is in accordance with the recommendations of the Khosla Committee.

Excessive constriction of bridge waterway results in afflux. In case of alluvial rivers, afflux causes bed scour which nearly compensates the waterway carrying capacity lost in constriction and hence afflux becomes negligible. When riverbed is non-scourable or can only be partially scoured, afflux cannot get obliterated and constriction of Lacey width may not be justifiable.

CHENAB RIVER
AT SHER SHAH

IN 1890 - 91 AFTER BRIDGING AND TRAINING

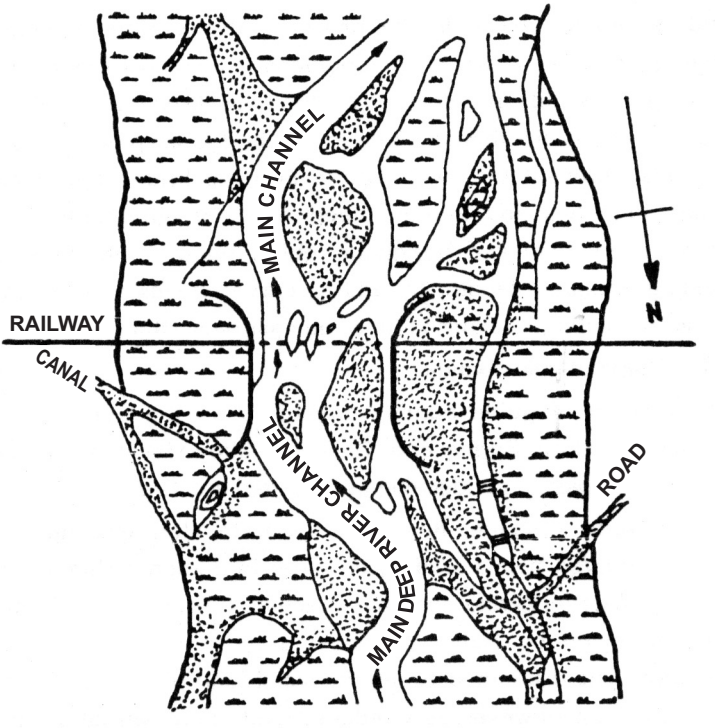


Fig. 5.1 : Railway bridge on Chenab river at Sher Shah

5.3 GUIDE BUNDS

Constriction of waterway at bridges is effected by extending the approach banks to cover spill area or, in addition, part of active channel. Such encroachment causes obstruction to flow. Guide bunds are then required to be provided for guiding flow smoothly through the bridge.

The course of main channel may change on account of progressive movement of meanders or formation and movement of islands in braided rivers. In the process, attack may develop on bridge approaches. Under such conditions it becomes obligatory to provide guide bunds in order to pass the river discharge axially with as uniform flow distribution through the bridge as possible. Guide bunds also prevent the river cutting into the bridge approaches, causing breaches and forming deep scour at abutments. These objectives are achieved by giving guide bunds suitable shape, providing adequate length, by adding curved heads and by giving suitable protection to exposed faces against river attack.

5.3.1 Shape of guide bunds

Alignment of guide bunds converging on the upstream was considered by Bell, Spring and Gales to allow for the waterway obstructed by piers on bridge line as indicated in Fig. 5.2 (B).

With such a shape, flow may separate after passing round upstream curved heads leading to shoal formation on flanks near the bridge axis rendering end bays inactive. Divergent guide bunds were conceived by Haigh and Spring to mitigate tendency of separation. The distance between the worst loop of the river and the approach bank however gets reduced and shoal formation in the centre in the wide section some time occurs as in Fig. 5.2(A). Symmetrical and parallel guide bunds were proposed by Spring and Sethi as depicted in of Fig. 5.2(C). When provided with upstream curved heads of sufficiently big radius, this shape of guide bunds is found to prevent separation of flow along flanks and help in effecting equitable distribution of discharge across the bridge section. It is also suitable under condition of variable approach. Elliptic guide bunds were advocated by Sharma et. al.[5.6] of the Uttar Pradesh Irrigation Research Institute (U.P.I.R.I.)

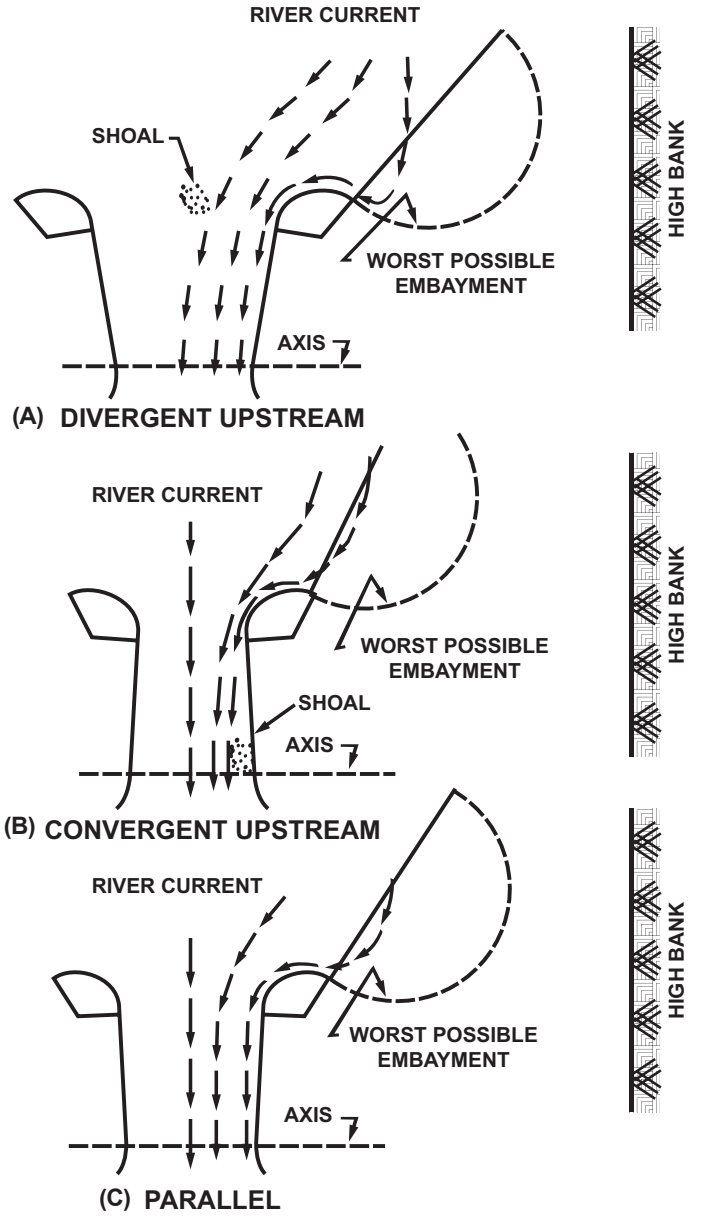


Fig. 5.2 : Guide Bunds of Differentshapes

in case of wide and shallow rivers to induce the flow to hug the guide bunds better without separation all along their lengths as shown in Fig. 5.1. Considering all the above shapes, in case of alluvial rivers with sandy bed and meandering pattern, elliptical shape appears preferable to minimise obliquity and separation of flow. Straight parallel guide bunds with composite curve for upstream heads have also been found to give optimum design as in case of Mokameh bridge on Ganga river. In case of rivers in submontane region, in rivers with bed material coarser than sand size and in rivers with braided channel pattern, experience is that nonsymmetrical guide bunds with different length· and shape on left and right banks may be warranted on account of local conditions and may be found to result in superior hydraulic performance.

5.3.2 Length of guide bunds

Length of a guide bund has to be such that the flow approaching even from behind the guide bund would pass through the bridge with minimum obliquity ensuring optimum utilisation of waterway. Length has also to be adequate to keep worst embayment of the river at the back of the guide bund away from the approach bank at a safe distance.

For coaxing the river to flow axially through the bridge lengths of guide bunds considered essential by Spring, Gales and Sethi are given in Table 5.1. Recommendations of Gales are for limiting obliquity of flow through the bridge from 30^0 to 34^0.

In addition, bend flow analogy is relevant in this case. The analogy of flow, downstream of a curved channel indicated in Fig. 5.3 suggests that flow distribution within guide bunds would become more uniform if radius of curved head is increased rather than increasing the length of the guide bunds as was exemplified by the left guide bund of the Kosi barrage.

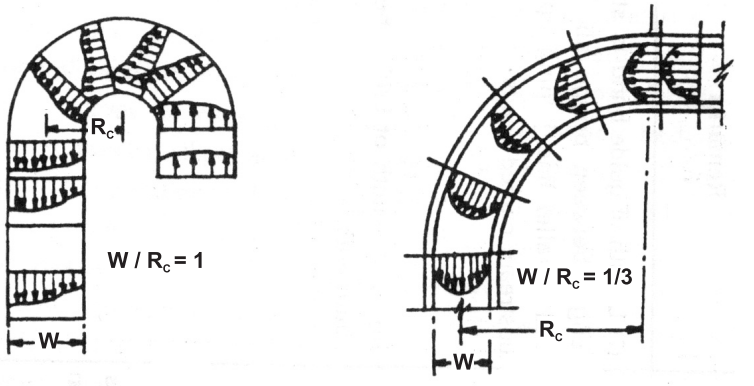


Fig. 5.3 : Velocity distribution in channelsof different curvatures

Table 5.1 : Length of Guide Bund

Portion	According to Spring	According to Gales	According to Sethi	Remarks
Up Stream Length	$1\frac{1}{10}$ L	$1\frac{1}{4}$ L for discharge of 7086 to 21254 m^3/s	9/10 to $1\frac{1}{10}$ L in flood plains and longer as required for rivers in submontane region.	(i) Length of guide bund is straight length between bridge alignment and parallel line joining apex of upstream curved heads. (ii) 'L' is length of bridge between abutments.
Down Stream Length	$^1/_{10}$ L to $^1/_5$ L	¼ L for discharge of 7086 to 21254 m^3/s 3/8 L for discharge of 42507 to 70847 m^3/s	$^1/_5$ to ¼ L for rivers in flood plains and longer as equired for rivers in submontane region.	

Procedure advocated by Spring, Gales and U.P.I.R.I. for determining length of guide bund to keep away the worst bend forming in the river was to first ascertain dimensions of such a bend obtaining in the river when a cut off occurs. Ratio of bend to chord at this stage in lower Ganga river was found by Gales to be 1.75. When such acute bends are not found in available surveys, radius of worst loop can be estimated on basis of average radius of existing bends. For each bend, the meander length ML along valley and meander width MB across valley is measured. Radius of curvature R can then be roughly worked out assuming the meander shape to have formed of circular arc and using the relationship.

$$(0.25\ ML)^2 = (MB - W)\ [R - 0.25\ (MB - W)]$$

wherein W is the average width of main channel during floods. Mean of the R values of all bends is found which then permits estimation of radius of worst bend using the ratio of R for average bends to R of worst bend varying inversely as discharge from 2.5 to 1.7 for discharges between 2000 to 9000 m^3/s as found at the U.P.I.R.I[5.7]. Garg and others from the U.P.I.R.I. proposed fitting such a curve behind guide bund when Khadir width is small and double bend when Khadir width is big. The procedure of fitting in worst bend at the back of the upstream of guide head bund is illustrated in Fig. 5.4.

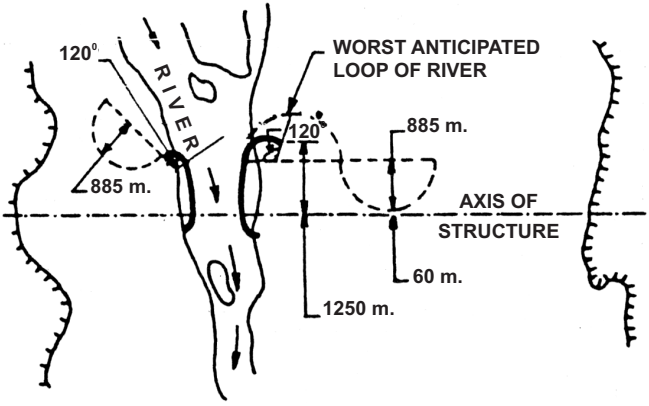


Fig. 5.4 : Fitting of worst bend at curved head of a guide bund

Length of guide bund determined on basis of the above two considerations of axial flow through the bridge and keeping the worst bend away at a safe distance may still be found to be inadequate to protect the long approaches in rivers having wide khadirs. Such a situation may be met with more frequently in case of braided rivers. Instead of elongating the guide bunds further, it would be advisable in such rivers to provide additional training works such as spurs, revetments etc. for protection of approach banks.

5.3.3 Curved heads

Several considerations govern the radius of curved head. More the radius and sweep, the cost becomes more. Minimum curvature necessary for running railway line to carry stone and construction materials is another important consideration. For broad gauge, curve having radius of 250 m is found to be suitable. Minimum curvature necessary to prevent separation of flow from the curved head has also to be determined and provided. Otherwise separation eddies would form deep and dangerous scour along the guide bunds.

According to Spring, water with a velocity of 2.4 to 3.0 m/s obtaining during floods can easily follow along a curve of 183 to 244 m radius. For lesser velocities radius can be reduced according to Table 5.2 as suggested by him.

Garg and other from the U.P.I.R.I. evolved, on the basis of model studies pertaining to 16 different structures, the following relationship for radius of curved head R_c (5.7) in m,

$$R_c = 0.45\ P_w$$

wherein P_w is the width in m given by Lacey Formula.

In case of Mokamah bridge over Ganga river, composite curve was evolved on basis of model studies. For the upstream head, 2^0 and 3^0 curves were provided upto 90^0 sweep and 5° curve between 90° and 120^0 where velocities were relatively lower as shown in Fig. 5.5 (Degree of a curve is obtained by dividing 1750 by its radius in m).

Table 5.2

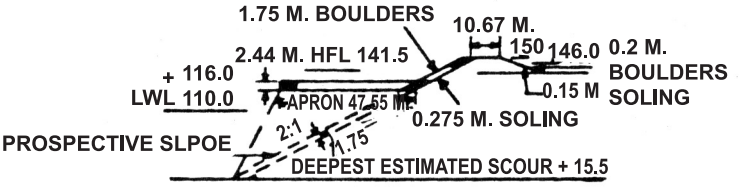
Radius of Curved Head According to Spring

Sand Size	Probable maximum abnormal scour below bed level		Radius of upstream curve head of guide bund (m) for Average fall of river in cms/km					Remarks
			4.66	9.32	13.98	18.64	27.96	
Very Coarse	Under	6.10 m	61	76	91	107	122	Radius of downstream curved head should be half of that of upstream curved head subject to a minimum radius required for maintenance trains to be shunted
	Over	6.10 m	76	95	114	134	152	
Coarse	Under	9.15 m	91	110	130	149	168	
	Over	9.15 m	107	131	156	180	204	
Medium	Under	12.20 m	122	130	168	191	213	
	Over	12.20 m	137	168	198	229	259	
Fine	Under	15.25 m	152	180	206	232	259	
	Over	15.25 m	183	221	252	282	311	
Very Fine	Under	18.30 m	183	213	244	274	305	
	Over	18.30 m	244	274	305	335	306	

Recommendations in respect of upstream and downstream curved heads and their angle of sweep made by Spring, Gales and Sethi are summarised in Table 5.3.

Direct comparison of radii of curved head obtained by following different practices is not possible. With certain assumptions and approximations, it was however possible to derive the following three relationships. (5.8)

$R_c = 71.80\ Q^{0.129}$ on basis of Spring's Table
$R_c = 0.935\ Q^{0.575}$ on basis of Gales and Sethi recommendations.
$R_c = 2.4\ Q^{0.5}$ on basis of recommendation of Garg et al of U.P.I.R.I



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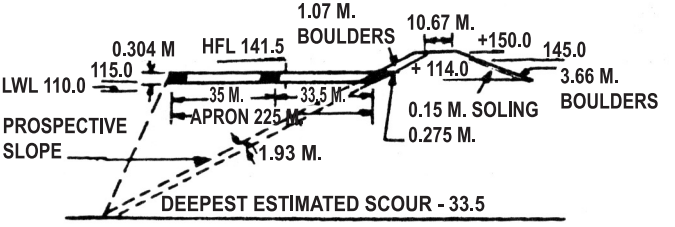

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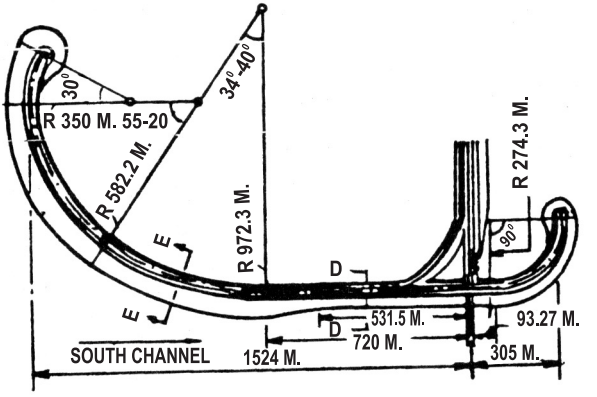


Fig. 5.5 : Composite curve for guide bund head of mokame bridge on Ganga river

Table 5.3

Radius of Curvature and Angle of Sweep According to Spring, Gales and Sethi

Item	According to Spring	According to Gales	According to Sethi
Radius R in m of upstream curved head	R = 183 to 244 m for V = 2.4 to 3.0 m/s, Radius depends on slope, scour depth and size of bed material as in Table 5.2 maximum radius being 366 m.	R = 249.6 m i.e. 7 degree curve for Q = 7086 to 21254 m^3/s and R = 582.2 m i.e. 3 degree curve for Q = 42507 to 70847 m^3/s	R = 249.6 to 582.2 m with composite curvature
Radius R in m of downstream curved head	(i) 61 to 91 m (ii) Half of upstream head (iii) Minimum required for shunting maintenance trains		R = 249.6 m i.e. 7 degree curve
Upstream head angle of sweep	120° to 140°	120° to 140°	120° to 140°
Downstream head angle of sweep		60°	60° (and not 90°)

Radius of curvature arrived at using these relationships for various discharges are given in Table 5.4.

Table 5.4 : Radius of Upstream Curved Head of Guide Bund According to Various Investigators

Discharge Q m^3/s	Radius of Curvature R_c in Metres according to		
	Spring	Gales	Garg
283	148.73	24.02	40.37
1416	183.07	60.63	90.31
2832	200.19	90.32	127.72
5564	218.41	133.17	179.02
7080	225.31	152.96	201.94
14160	246.38	227.86	285.59
21240	259.61	287.69	349.77
28321	269.43	339.45	403.89
42481	283.90	428.57	494.66
56641	294.63	505.66	571.18
70802	303.24	574.89	638.61

For a discharge of 14160 m^3/s, Spring recommended radius of about 247m. Gales and Garg also give comparable values of 228 and 284m respectively. However, for small discharges of the order of 283 m^3/s Spring's value of Rc is more than six times Gale's while for high discharge of 70802 m^3/s, Spring's value is almost half of that given by Gales and Garg. At the present stage confirmatory and conclusive evidence for supporting anyone of the above practice is not available. It is, therefore, advisable to follow Spring's Table for discharge upto 14160 m^3/s and Garg's recommendation for higher discharges.

5.3.4 Protection for side slopes

Guide bund is normally constructed with sand core with side slopes of 2: 1 and provided with stone protection on river side which is continued along the back of curved heads., Clay should not be used for constructing guide bunds since it is subject to heavy settlement and clays soluble in water are liable to be sucked out by the current. On the backside of the shank turfing on clay blanket is given. The material for core should be obtained from the riverside and not from the back side. The weight of stone according to Spring should be between 25 and 50 kg. He observed that such stones, called one man stones, can withstand velocities up to 5.5 m/s. Stones should be angular and not rounded and should be durable.

Thickness of stone protection recommended by Spring is given in Table 5.5.

Table 5.5

Size of riverbed material	Thickness of stone protection (cm) for average fall of the river (cm/km)				
	5	14	19	28	37
Very coarse	40	47.5	55	62.5	70
Coarse	55	62.5	70	77.5	85
Medium	70	77.5	85	92.5	100
Fine	85	92.5	100	107.5	115
Very Fine	100	107.5	115	122.5	130

He suggested that thickness can be reduced by 150 to 225 mm by using quarry refuse or burnt bricks as a filter. The thickness was recommended to be increased by 25 percent at head to take care of severe attack and 25 percent all over when pitching is dropped through deep water below low water level.

Thickness of pitching and soiling for guide bund slope recommended by Gales is given in Table. 5.6. Soling formed by stone ballast to act as filter was proposed by him.

Table 5.6
Thickness of Stone and Soiling for Slope for Guide Bunds According to Gales

Item	River Discharge (m^3/sec)						Remarks
	7086 to 21254		21254 to 42507		42507 to 70847		
	U/S head	Body and D/S head	U/S head	Body and D/S head	U/S head	Body and D/S head	
Thickness of pitching stone (cm)	105	105	105	105	105	105	Pitching stone to be hand set.
Thickness of soiling ballast (cm)	17.5	17.5	20.0	20.0	22.5	22.5	Ballast to be broken to pass 6.25 cm ring
Total thickness of slope covering (cm)	122.5	122.5	125	125	127.5	127.5	

Inglis[5.9] proposed that thickness of pitching be made dependent on discharge such that

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

wherein T is thickness in m and Q the discharge in m^3/s.

Sethi suggested that thickness should be varied in accordance with discharge as well as sand size following the relationship.

$$T = K\left(\frac{Q}{F}\right)^{1/3}$$

wherein K is the coefficient varying inversely with the discharge. This variation was given graphically as shown in Fig. 5.6 and in tabular form as in Table 5.7.

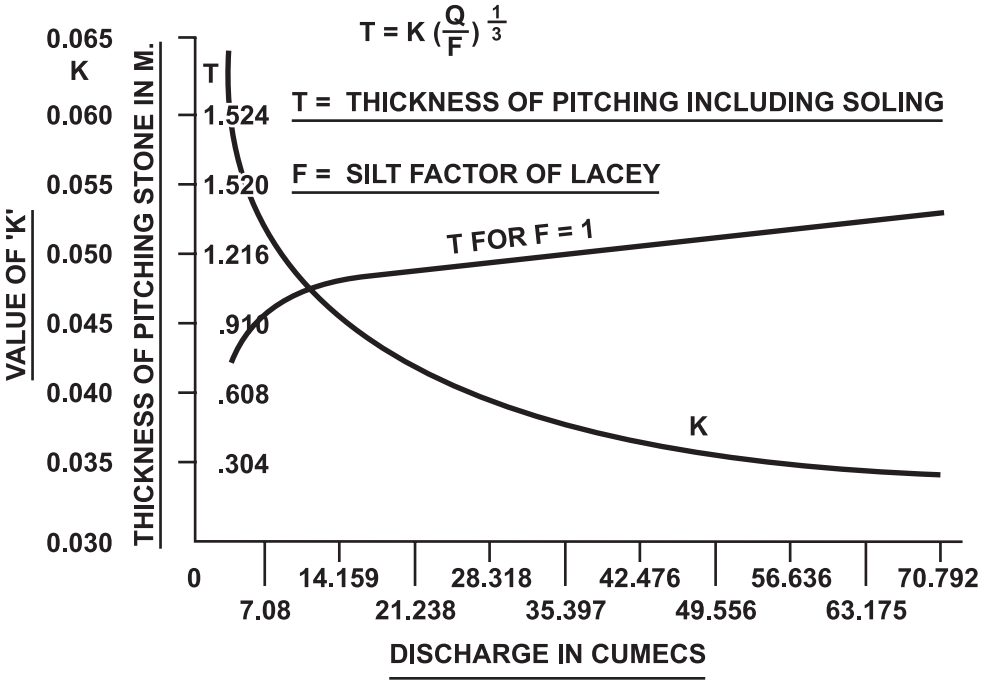


Fig. 5.6 : Thickness of pitching recommended by Sethi

Table 5.7 : Thickness of stone pitching for shank according to Sethi

S.No.	Discharge (m^3/sec)	Soiling thickness (cm)	Thickness of Pitching Stone (cm)	Total Thickness 'T' (cm)	Remarks
1	2832	15	75	90	Values in this Table were worked out using the relationship $T=K(Q/f)^{1/3}$ and adopting values of K for various values of Q from figure 5(b) given by Sethi in Reference 4. Thickness of stone pitching in underwater slope should be 1.33 T for shank and downstream head. Use of 45 to 60 cm thick cement concrete slabs on slopes supported by a suitable toe may be considered in place of pitching stone where costs of the two arrangements compare favourably. Thickness of stone pitching for U/S head above low water level should be same as that of shank. For underwater slope, the thickness for U/S head should be 1.5 T.
2	7079	15	82.5	97.5	
3	11327	15	90	105	
4	14159	15	97.5	112.5	
5	28317	15	105	120	
6	42475	22.5	105	127.5	
7	56634	22.5	112.5	135	
8	70790	22.5	120	142.5	

Thickness for underwater slope was stipulated to be' 1.5 T for head and 1.33 T for the body and tail.

Considering practices advocated by different authors, one man stones of 30 cm to 40 cm size weighing 40 to 70 kg are considered adequate to withstand velocities up to 3.5 m/s which are commonly met with in alluvial rivers in flood plains. The thickness of pitching is, however, varied to meet different situations. Spring proposed variation in thickness according to river slope and size of bed material, according to possibility of providing filter, according to difficulties in achieving proper standard of work as for underwater placement and according to exposure to attack as at curved head. Gales and Inglis proposed variation in thickness according to discharge while Sethi considered size of bed material as additional relevant parameter.

Apart from the above Indian practice, theoretical approach to the problem of design of pitching is also possible. Stone can get dislodged due to hydrodynamic drag and lift caused by velocity of flow. For preventing dislodgement either by sliding or by overturning, the size of stone is required to be varied as the square of the velocity. Size or weight necessary for stability of an isolated stone is much more than in case of stone surrounded by others when laid to form a layer. Incidence of direct attack caused by oblique flow, formation of eddies, higher level of turbulence are some other factors requiring heavier stones for stability against a current of given velocity. For the same velocity, rounded stones, stones with higher vertical dimension, stones with smaller specific gravity and stones on steeper side slope are less stable. For stability of stone pitching against current velocity on a side slope, the slope angle has to be flatter than the angle of repose of pitching material. On account of such numerous factors governing stability of stone, value of C in various relationships of the form

$$\text{Velocity } V = C_1 \text{ (diameter } D)^{1/2}$$

can be different. Since volume and weight of a spherical body are interrelated, this relationship can as well be expressed in terms of weight, Weight $W = C_2$ (velocity $V)^6$

wherein W is the weight of a sphere having the same volume as

that of the stone and C_1 and C_2 are coefficients. Some such relationships are presented in Table 5.8.

Normally the side slope of guide banks is kept at 2 horizontal to 1 vertical. For this side slope, relationship recommended by the Indian Standards Institution in IS 8408-1976 for protection of side slope is

$$W = 0.02\ V^6$$

which is shown plotted in Fig, 5.7 reproduced from Fig. 6 in IS : 8408-1976.

According to railway practice one man stone used for protection works has a weight of about 55 kg (120 lbs) and can withstand velocities up to 3.5 m/s (11.5 ft/*s)* or so. IS: 8408:1976 also states that one man stone weighing 40 to 70 kg (giving mean weight of 55 kg.) can withstand velocities upto 3.5 m/s. For W of 55 kg and Y of *3.5* m/s*,* the relationship of the form $W = CV^6$ gives as the value of C as 0.03.

In IS: 8408-1976, thickness of pitching stone given in clause 5.5 is equal to size of stone but not less than 0.2 m. In railway practice, the thickness of pitching stone based on recommendations of Spring, Galses, Sethi, Inglis, etc. is much more in vogue. If thickness is to be reduced to conform to ISI specifications, the size or weight has to be bigger. In that case the coefficient should be even more than 0.03, say equal to 0.04, giving

$$W = 0.04\ V^6$$

in preference to relationship

$$W = 0.02\ V^6$$

recommended in IS : 8408-1976 for determining size and weight of stone on the slope of the guide bunds.

Regarding apron protection two limiting curves are given in Figure 7 of IS: 8408-1976 which is reproduced as Fig. 5.8, depicting relationships

$W = 0.0161\ V^6$ for surrounded stone, and
$W = 0.1003\ V^6$ for isolated stone.

Table 5.8 : Relationships for Size and Weight of Stone

S.No. (1)	Relationship for Size (2)	Relationship for weight (3)	Sponsoring Agency (4)	Source (5)	Remarks (6)
1.	$V_b = 5.95D^{1/2}$ (V_b is velocity against stone)	$W = 0.031\ V_b^6$	U.S. Bureau of Public Roads	Adopted from the report of Sub-Committee on slope protection in A.S.C.E., proceedings, June, 1948	Relationship is for 2:1 side slope. T.V.A. adopted similar procedure vide reference 5.11. Depending on the severity of attack V_b is increased up to 100 percent
2.	$V_m = 4.2\ D^{1/2}$ (V_m is local average velocity)	$W = 0.254\ V_m^6$	U.S. Army Corps of Engineers	Relationship originally given in Ref. 5.12 is $T=0.04\ (r_s-r)\ D_{50}$. This was converted by author as $V_m = 4.9\ D^{1/2}$ and then side slope effect allowed for as suggested in Reference 5.12 which gave $V_m = 4.2\ D^{1/2}$ for 2:1 side slope	Relationship is for 2:1 side slope and derived from relationship in terms of T given for horizontal bottom.
3.	$V_m = 5.65\ D^{1/2}$	$W = 0.043\ V_m^6$	United States Bureau of Reclamation curve.	The curve is given in Hydraulic Design Criteria of U.S. water-ways experiment station, in Ref. 5.14, sheet 712-1. It approximates Isbash Curve with E = 0.86	Meant for protection of channel bottom and side slopes down-stream of stilling basins and for rock sizes for river closures. V_m in preference to V_b is recommended to be adopted to account for indeterminate flow factors like obliquity of attack, eddy action, etc. It nearly corres-ponds to curve of Bureau of Public Roads with 1:1 side slope.

S.No. (1)	Relationship for Size (2)	Relationship for weight (3)	Sponsoring Agency (4)	Source (5)	Remarks (6)
4.	For SS 1:1 $V_m = 5.00\ D^{1/2}$ For SS 2:1 $V_m = 6.37\ D^{1/2}$ For SS 3:1 $V_m = 6.81\ D^{1/2}$	For SS 1:1 $W = 0.089\ V_m^6$ For SS 2:1 $W = 0.021\ V_m^6$ For SS 3:1 $W = 0.014\ V_m^6$	California Public Works Department.	IS : 8408-1976 and Draft of 'Criteria for Design of Guide Banks for Alluvial Streams' prepared by BDC.68 of I.S.I.	The plots in figure 6 of IS 8408-1976 are based on the relationship $W = \frac{0.1136 V^6 S_S . Z}{(S_S - 1)^3}$ wherein Z is $Cosec^3\ (70^o - a)$, S_s is specific gravity of stone, a is angle in degrees of side slope. This relationship was adopted from California Public Works Department and shown plotted in Fig.6 without giving relationship. They are recommended by I.S.I for slope protection. CPW Department recommends V_m to be increased by 33% for very violent attack and reduced by 33% for practically no attack.

S.No. (1)	Relationship for Size (2)	Relationship for weight (3)	Sponsoring Agency (4)	Source (5)	Remarks (6)
5.	$V_m = 6.65\ D^{1/2}$ for surrounded stone. $V_m = 4.9\ D^{1/2}$ for isolated stone	$W = 0.0161\ V_m^6$ for surrounded stone, $W = 0.1003\ V_m^6$ for isolated stone.	ISI	IS 8408-1976 and Draft of 'Criteria for design of Guide Banks for Alluvial Streams' prepared by BDC 68 of ISI.	ISI has recommended these relationships for apron protection. These two relationships are shown plotted in Fig. 7 without giving relations. They are supported by following relationships. $V = 6.5\ D^{1/2}$ - Garde, $V = 6.8\ D^{1/2}$ - Isbash for surrounded stone and $V = 4.9\ D^{1/2}$ - Isbash, $V = 4.9\ D^{1/2}$ - Berry, $V = 4.8\ D^{1/2}$ - Mavis Laushy for isolated stone.
6.	For 40 kg stone and 3.5 m/s velocity $V_m = 6.32\ D^{1/2}$ For 70 kg stone and 3.5 m/s velocity $V_m = 5.76\ D^{1/2}$ $V_m = 4.9\ D^{1/2}$ For 55 kg stone and 3.5 m/s velocity $V_m = 5.99\ D^{1/2}$	For 40 kg stone and 3.5 m/s velocity $W = 0.0218\ V^6$ For 70 kg stones 3.5 m/s velocity $W = 0.0381\ V^6$ For 55 kg stone and 3.5 m/s velocity $W = 0.0300\ V^6$	Indian Practice	IS 8408-1976	40 to 70 kg stone of 0.3 to 0.4 m diameter is said to withstand velocities upto 3.5 m/s

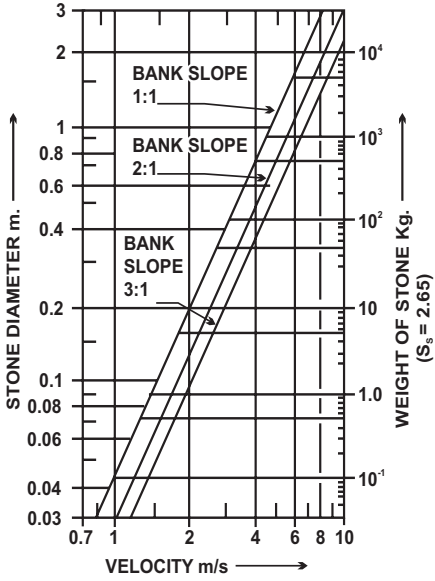


Fig. 5.7 : Size pitching stone vs velocity

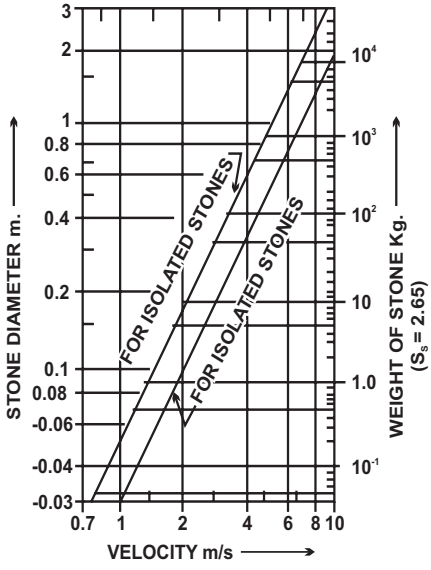


Fig. 5.8 : Size of apron stone vs velocity

The weight of stone when isolated is thus nearly six times the weight of surrounded stone. Instead of this extreme factor of 6, if factor of 3 is used, the relationship of apron stone worked out from relationship of slope stone would be

$$W = 3 \times 0.04\ V^6 = 0.12\ V^6$$

or nearly $$W = 0.1\ V^6$$

or $$V = 4.9\ D^{1/2}$$

or $$D = 0.042\ V^2$$

This is also the relationship for isolated stone given by the I.S.I., and hence appears acceptable.

The size and weight so determined can be then increased arbitrarily to account for direct impingement due to sharp change in approach direction, for increased severity of attack, degrees of turbulence, incidence of waves etc. The thickness of pitching is generally kept equal to and limited to twice the size of stone and for preventing sucking of underlying material of bank section through interstices, proper filter is provided in between.

Frequently part of the guide bund slope is required to be pitched under submerged condition below low water level when satisfactory provision of filter becomes infeasible. Control in placement of 1-2 layers of pitching stone under water also cannot be rigorous. Hence theoretical approach normally becomes difficult to follow for under water work in absence of special materials and equipment. The method can, however, be used when proper standard of work can be achieved, the entire or most of the slope pitching being in dry. If quality control is difficult, it is considered advisable to follow the Indian Practice mentioned before, of providing the thickness of pitching arbitrarily much more than 2-3 layers, following recommendations of Spring, Gales, Inglis etc., since it has withstood the test of time at bridges on all major river systems.

5.3.5 Apron protection

5.3.5.1 Scour depth

Apron is provided beyond toe of slope of guide bund so that when bed scour occurs, the scoured face will be protected by launching of apron stone. For, arriving at the quantity of apron stone probable depth of scour needs to be estimated. Lacey formula,

$$D = 0.473\ (Q/f)^{1/2}$$

wherein D is depth in m below H. F. L., Q is discharge in m^3/s and f is silt factor is used in this connection to work out flow depth and this depth is increased by using multiplying factors given in Table 4.3 to arrive at scour depth below H.F.L. Silt factor f is given as 1.76 $m^{1/2}$ m being weighted mean diameter of bed material in mm.

5.3.5.2 Scour factors

According to Spring. a fair basis to estimate probable scour depth along the guide bund can be the worst abnormal scour to be found in the river which is expected to be 2 to 2½ times the normal flood depth.

Gales proposed scour depth estimation to be based on observed scour along eroding bend and increasing this depth suitably for difference in severity of attack as on guide bund head, body or tail and other factors. Gales recommendations in this respect are reproduced in Table 5.9.

Table 5.9 : Estimation of Depth of Scour according to Gales

Item	River Discharge (m^3/sec)		
	7086 to 21254	21254 to 42507	42507 to 70847
Observed deepest scour below low water level along a soft cutting bank in the bend at ¾ falling flood.	x_1	x_2	x_3
Add 33 percent to convert these depths to those obtainable at a rigid bank	$0.33x_1$	$0.33x_2$	$0.33x_3$
Deepest known scour	$1.33x_1$	$1.33x_2$	$1.33x_3$
Percentage addition to deepest known scour to be made for contingencies such as unlikelihood of finding absolutely deepest scour, narrowing of the river & for severe attack on the guide bund head.			
For body and tail of guide bank	25%	32%	45%
For upstream head of guide bank	50%	63%	60%
Deepest known scour to be adopted below low water level			
For body and tail of guide bank	$1.66\ x_1$	$1.75\ x_2$	$1.93\ x_3$
For upstream head of guide bank	$2.00\ x_1$	$2.17\ x_2$	$2.53\ x_3$

Khosla proposed scour factors depending on the part of guide bund under consideration as in Table 5.10.

Table 5.10 : Estimation of Depth of Scour according to committee headed by Khosla

Location	Range of Scour Depth	Mean depth to be adopted
Nose of guide bank	2.0 to 2.5 D	2.5 D
Transition from Nose to Straight Portion	1.25 to 1.75 D	1.5 D
Straight Reach of Guide Bank	1.0 to 1.5 D	1.25 D

D is Depth below H.F.L. and proposed to be estimated using Lacey formula

$$D = 0.473\ (Q/f)^{1/3}$$

referred to before. If width of river channel is found to be less than that obtained by Lacey formula

$$W = 4.836\ Q^{1/2}$$

depth formula in terms of Q becomes inapplicable and the one based on discharge intensity was proposed to be adopted viz.

$$D = 1.34\ (q^2/f)^{1/3}$$

wherein q is the discharge intensity in m^3/s per metre width.

Inglis advocated that for large, radius nose of guide bund, scour should be estimated using the formula.

$$D = 2.75\ D_{Lacey}$$

implying scour factor of 2.75

Sethi gave different scour factors for different parts of the guide bund as in Table 5.11.

Table 5.11 : Estimation of Depth of Scour according to Sethi

Location	Scour Depth below h.f.l.
Upstream nose of curved head	2.75 D
Straight portion' of shank and tail	1.75 D
Portion of Shank opposite pier and 33 m on either side	2.0 D

Note: D is flow depth in metres according to Lacey formula as in Table 5.10

"Bridge Sub-Structure Code" of Railways[5.10] specified that depth of scour be estimated using the Lacey formula.

$$D = 0.473\ (Q/f)^{1/3}$$

The scour factor recommended for upstream curved head was 2.5 to 2.75 and for the rest of the guide bund portion 1.5.

Considering recommendation of various authors in respect of scour factors, provisions in 'Bridge Sub-structure Code' appear sufficiently conservative.

5.3.5.3 Apron quantity

In working out apron quantity, thickness of pitching is required to be increased to allow for difference in quality of work when pitching is laid in dry and pitching formed by launching under water after scour in river bed. Spring and Gales proposed increase in pitching on this account as well as on account of other factors as given in Table 5.12 and 5.13.

Table 5.12 : Thickness of Launched Apron according to Spring

S. No.	Item	For Shank	For Head
1.	Thickness of pitching stone on slope 'T'	As in Table 5.5	1.25 T
2.	Additional thickness for possible non-uniform launching: (a) in fairly steady rivers (b) in rivers liable to sudden deep scour	 25% 50%	 25% 50%
3.	Allowance for fanning at curved heads	-	As in Fig.5.9

Table 5.13 : Thickness of Launched Apron according to Gales

SN	Item	For River Discharge 7086 to 21254 m^3/s		For River Discharge 21254 to 42507 m^3/s		For River Discharge 42507 to 70847 m^3/s	
		Up-stream Head (cm)	Body and Tail (cm)	Up-stream Head (cm)	Body and Tail (cm)	Up-stream Head (cm)	Body and Tail (cm)
1	Thickness of pitching stone	105	105	105	105	105	105
2	Addition for absence of soiling at 33%	35	35	35	35	35	35
3	Addition at 11 and 22% for high discharge			11	11	23	23
4	Addition at 11 and 22% for high silt content			11	11	23	23
5	Addition at 22% for head	23		23		23	
6	Total thickness of pitching stone	163	140	185	163	209	186
7	Apron thickness = 1.5 times thickness of pitching stone over 2 horizontal : 1 Vertical launched face	245	210	278	245	315	278
8	Berm thickness	Same as apron Thickness					

At curved heads, the apron launches on a conical surface. Apron quantity is accordingly required to be increased suitably as in Fig. 5.9.

Volume of stone = 2.81 p D X T [r_1 + 2(F + R) + D]

This quantity is to be laid in an area

$$\frac{\pi}{2}\left(r_3^2 - r_2^2\right)$$

5.3.5.4 Apron as laid

Dimensions of apron according to Spring and Gales are indicated in Fig. 5.10(A) and 5.10(B). Gales and Sethi favoured uniform thickness of laid apron while Spring advocated tapered design. Spring's design has been found deficient near the toe and hence uniform thickness appears superior. Sethi has omitted berm but recommended extension of soling for 3 to 6 m beyond. Width of apron when laid equal to 1.5 times the depth of scour below riverbed has been found to permit satisfactory launching. Since apron is assumed to launch on a side slope of 2:1, the thickness of apron when laid is given by 'quantity of apron' / 'width of apron laid.'

5.3.5.5 Construction programme

Suitable phasing of construction of various components is essential for successful completion of the project and efficient functioning of the bridge. Construction of only part of the guide bunds or only one of the two guide bunds in one working season is considered inadvisable. Both guide bunds, complete with their curved heads should be completed simultaneously in one working season.

5.3.6 Provisions as contained in IRBM para 810 with regards to Guide Bunds

5.3.6.1 Necessity

Guide bunds are meant to confine and guide the river flow through the structure without causing damage to it and its

approaches. They also prevent the out flanking of the structure.

5.3.6.2 Shape and design features

a) The guide bund can either be divergent upstream or parallel. In the case of divergent guide bund, there is possibility of formation of a shoal at the center. Parallel guide bunds minimise obliquity and separation of flow along the flanks. According to geometrical shape, the guide bunds may be straight or elliptical. In the case of certain type of alluvial rivers with sandy bed and meandering pattern, elliptical shape appears preferable to minimise obliquity and separation of flow. Various types of guide bunds are shown in Annexure 8/3.

b) Normally the upstream shank of the guide bund is between 1.0 to 1.5 times the length of the bridge, while the downstream shank is between 0.25 to 0.4 times the length of the bridge.

c) The tail bund on the downstream side is provided to afford an easy exit to the water and to prevent formation of vertical whirlpools or rollers which give rise to scour. These tail bunds are also curved at their ends and should be properly armoured.

d) The guide bund is provided with a mole head on its upstream side. The mole head bears the brunt of the attack and should be provided with adequate protection in the form of slope pitching and properly designed launching apron. The shank i.e. the portion behind the curved mole head of the guide bund should also be similarly protected on the river side. The slope in the rear of the guide bund need not necessarily be provided with pitching and may be protected by planting grass or shrubs as found suitable.

e) Radius of curved upstream mole head may be taken as 0.45L (L is water way width determined from Lacey's formula subject to minimum of 150m and maximum of 600m). The radius of downstream curved tail may be kept as 0.3 to 0.5 times the radius of upstream curved head. The angle of sweep of curved head may range from 120^0 to 145^0 according to river curvature and that of the tail head may be kept as 45^0 to 60^0.

For smaller rivers, one single radius is good enough. Forimportant rivers, multi radii may beselected generally after model studies for smoother flows

.

f) Top width of the shank of the guide bund should be wide enough to permit plying of trucks and keeping reserve boulders for maintenance. From this consideration top width may be taken between 6m to 9m, and side slopes may be taken as 2:1.

g) Side slopes of guide bund needs protection on following counts:-

i) Wave action on the upstream side
ii) Water current along the slopes
iii) Wind action
iv) Rain cuts/Rain water

Most common method is to provide stone pitching. It is necessary to provide 20 cm to 30cm thick graded filter below the pitching. Stone used for pitching is generally man size boulder of 35 to 55kg so that they cannot be easily displaced by the current. For small works, one stone thick pitching (25 to 30cm) should suffice. Gaps in between could be filled up by smaller pieces.

In case of guide bund, the pitching should continue right up to the top of the formation for the river side, including the curved head on both sides and tail head. For important rivers or in case of large ponding etc, the pitching should be done on the rear side of the guide bund also. For approach embankment, on the upstream side, the pitching should continue up to the free board level which should be determined not only on HFL but also to take care of velocity head ($V^2/2g$), wave action etc. For the downstream side, pitching may be done up to the water level based on hydraulic model study or general water level observed.

A good drainage is key for protection of slopes from rain cuts, particularly on high banks of over 6m height. For this,longitudinal and cross drains should be provided.

Guide bunds and approach embankments particularly in

khadir of the river must be constructed in one go in one season. In case this is not possible, at least, a wedge size equal to angle of internal friction of the old construction should be removed and the next construction should be done with proper benching. For slope protection and apron, an overlap may be provided.

h) No spurs projecting from the guide bunds should, in any case, be provided.

j) For design and construction of guide bunds/launching aprons reference may be made to IS: 10751-1994 (Planning and Design of Guide Banks for Alluvial Rivers – Guidelines) and IRC: 89-1997 (Guidelines for Design and Construction of River Training and Control Works for Road Bridges)

5.3.6.3 Apron Protection for guide bunds

a) Apron is provided beyond the toe of the slope of the guide bund, so that when bed is scoured, the scoured face will be protected by launching of the apron stone or wire crate containing stone.

b) Following are the important details for design of apron:

i) Thickness of apron

Thickness of apron is governed by thickness of pitching on the slopes of the guide bund (T). In case of straight portion of guide bund, the thickness of apron through its width is generally kept as 1.5T. In case of curved portion of guide bund, the thickness of apron is generally kept as 1.5T at the junction of apron with pitching on the slope and the same is increased through its width to 2.25T at the end of apron.

ii) Level at which the apron is to be laid Normally apron should be laid on dry bed, as low as possible.

iii) Width of apron

Width of apron is determined by depth of scour and is generally

kept as 1.5 times the differen ce between the deepest known scour level and low water level.

5.3.6.4 Maintenance:

a) Substantial reserve of pitching stone should be maintained on the guide bund for use during emergency. This should be stacked at the top of the guide bund. Quantity of reserve stock to be maintained at guide bund should also be specified by Principal Chief Engineer/Chief Bridge Engineer.

b) The track on the guide bund, where provided, should be maintained in a satisfactory condition and should be capable of taking boulder trains at any time. The Permanent Way Inspector and the Assistant Engineer should inspect the track soon after the monsoon every year and carry out necessary repairs well before the next monsoon.

c) Every effort should be made to ascertain whether the apron is launching to the intended position and this should be done by probing after the flood season is over. Plotting of the levels will indicate the efficacy of the launching.

d) Disturbance of pitching stone on the slope indicates dangerous condition and additional stones should be placed in position immediately as necessary.

5.3.6.5 Failures and remedial measures:

The conditions under which an apron of the guide bund can fail and remedial measures to be adopted are stated below:

a) If the launching takes place beyond the capacity of the stone in the apron and results in leaving the bank material exposed to the current and wave action, more stone will have to be added to the apron.

b) If stones are carried away by high velocity current from the launching apron and the toe of the bund, the apron should be strengthened against severe attack by laying large sized stones at the outer edge of the apron.

c) If slips and blow -outs in the bund occur due to a steep sub soil water gradient resulting from a rapidly falling flood in the river, the bank should be widened to reduce the hydraulic gradient. This equally applies to marginal bunds.

d) Wherever disturbance is noticed in rear of guide bund due to wave lash or other causes, the slope pitching should be adopted as a remedial measure.

e) An apron can launch satisfactorily only if the material scours easily and evenly and the angle of repose of the underlying material is not steeper than that of the stone. In all these cases action should be taken to dump the boulders on the toe of the bank and make up irregular surface.

5.4 SPURS OR GROYNES

5.4.1 Functions

Spurs or groynes are structures constructed in the river transverse to the bank to achieve any of the following objectives.

Deflecting spurs are commonly used for protection of riverbank from erosion either in a straight or a curved reach. When navigation channel has inadequate depth, spurs are provided to constrict the channel width so that depth is increased suitably by bed scour. Closure of bye channels may be required to be made in the interest of navigation or as a river training measure. It is possible to achieve this purpose by means of spurs. In the approaches to bridges and water intakes, shifting position of the channel is some times required to be stabilised. Such stabilisation can be effected up providing attracting spurs.

5.4.2 Types

Spurs can be of varied types. They may be either permeable or impermeable. Permeable spurs are open structures constructed by driving wooden balties filled in with brush wood and weighed down by stone. When concentration of suspended sediment load is heavy, permeable spurs cause quick siltation due to damping of velocities. Such spurs are thus helpful in protecting the bank by forming sediment berms along the toe.

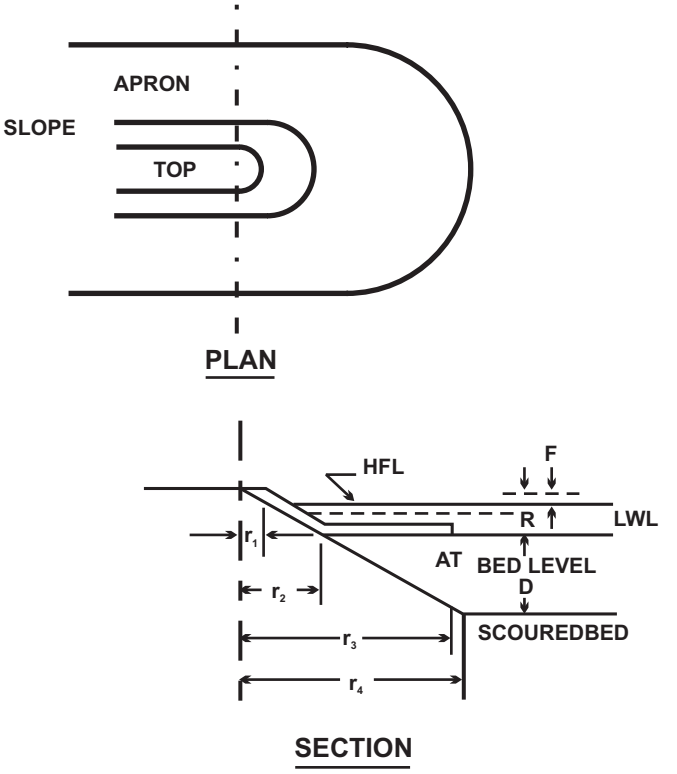
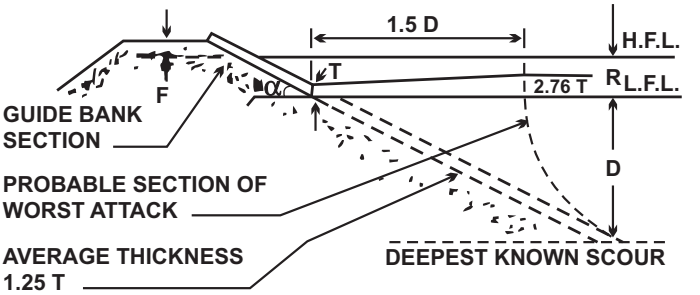


Fig. 5.9 : Apron for curved headof aguide bund with allowance for fanning out



ACCORDING TO SPRING

F = Free board.

R = Rise of flood.

D = Deepest known scour.

T = Thickness of slope stone.

Area of slope stone = 2.25 T (R+F) ie. T (R+F) Cosec α

Area of apron stone = 2.82 DT ie. TD Cosec α

Width of apron = 1.5 D.

Mean thickness of apron = 1.88 T.

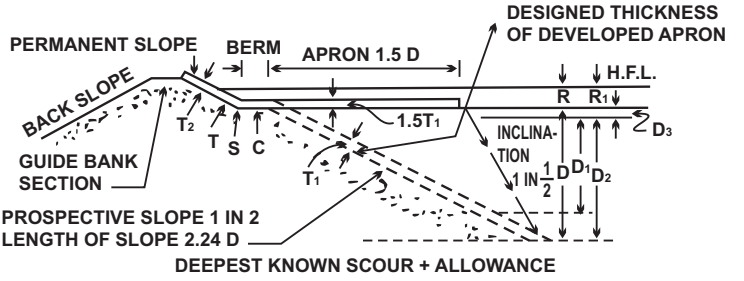
Inside thickness of apron = T.

Outside thickness of apron = 2.76 T.

Inclination of slope stone = 2 to 1.

Desired inclination of apron stone = 2 to 1.

Fig. 5.10 (A) : Apron as laid according to spring



ACCORDING TO GALES

F = Free board.

R = Rise of flood above bottom of apron.

D = Depth of scour for calculation of apron stone.

T = Thickness of permanent slope stone.

S = Thickness of soling.

T_2 = Thickness of covering T + S.

C = Thickness of clay covering.

T_1 = Thickness of stone on prospective slope bottom of apron.

W = Width of berm = 15 for class A Rivers.
= 20 for class B Rivers and
= 25 for class C Rivers.

Area of permanent stone = 2.24 (R+F)

Area of prospective slope stone = 2.24 D x T_1

Area of berm stone = W x 1.5 T_1

Width of apron = 1.5 D.

Thickness of apron = 1.5 T_1.

Area of apron stone = 1.5 D x 1.5 T_1

In construction abrupt changes in the width of the apron should be avoided.

Back slope to be suitably protected by stone pitching or grass.

REFERENCES :

Class A Rivers - Q between 2.5 and 7.5 lakh Cs.

Class B Rivers - Q between 7.5 and 15 lakh Cs.

Class C Rivers - Q between 15 and 25 lakh Cs.

Fig. 5.10 A : Apron as laid according to gales

Impermeable spurs are made of solid core with exposed faces protected by pitching. Such spurs can withstand severe river attack better than permeable spurs. Deposition occurs on the downstream side of the spur due to material scoured from the nose and the high velocity current gets deflected away. Solid spurs are accordingly provided where attack of flood flow is to be diverted. According to height, the spurs are classified as full height spurs when their top level is designed to remain above the highest flood level. On the other band, when top level is lower than the high flood level, spurs get occasionally submerged. Such spurs are called part height spurs. Short submerged spurs with height not exceeding 3 m or so when suitably aligned with respect to flow direction are effective in protecting the bank and are termed as bed bars. Another scheme of classification is according to orientation of the spur. Spurs may be aligned with respect to the flow direction, facing either upstream, normal or downstream. Spurs facing upstream are termed deflecting or repelling whereas spurs facing downstream are called attracting spurs. The term attracting is used in a restrictive sense. Attracting action of the spur can only be limited and that too under favourable conditions. Spurs aligned normal to direction of flow are called normal spurs. According to the shape of the spur head, the spur is termed a T-headed spur, a hockey stick spur, a curved headed spur or a round nosed spur. Material of construction is still another criterion used in differentiation of spurs. Spurs may be either stone spurs, brick spurs, bally pile spurs, spurs constructed of stone crate, concrete blocks or trees.

5.4.3 Location

Spurs are required to be properly located to achieve the desired purpose. When they are constructed for bank protection, eroding reach needing such protection is required to be determined by study of successive bank lines. In straight reaches of rivers, bank erosion may be caused by excessive velocities during high flood, obliquity of flow or squeezing of channel section by moving islands in a braided river. In meandering rivers, erosion is caused due to increase of depth, steepening of side slope and consequent instability of bank along concave side of a bend, which results in progressive erosion.

5.4.4 Length, spacing, inclination and height

Minimum length of a spur should be such that extent of scour hole at the nose as well as embayment in between the spurs do not reach the bank. Extent of scour hole in plan in the direction towards the bank can be assumed to be 2.75 times the scour depth D Lacey. Estimation of scour depth is dealt with in the Chapter 6. The embayment in between the spurs depends on spacing of spurs, bank alignment and orientation of spurs. In a straight channel, for normal spurs, the embayment is roughly equal to length of the spur when spacing between the spurs is three times the spur length, as has been observed in Kosi river downstream of Bhimanagar barrage. In an acute bend, embayment equal to length of the spur is obtained with spacing of spurs roughly equal to twice the spur length, as observed in Ganga river in acute bend at Mansi. If spurs are too long, the constriction of river section may become excessive and the attack on spurs can be severe. In such a case the velocities and channel depth on the opposite bank may also get affected. In any case extending the spur length to obstruct deepest course carrying main flow of the channel is generally hazardous. Channel obstruction beyond 20 per cent of the width is normally avoided. Length and spacing are related to economics, the nose portion costing heavily. Fewer number of spurs with required longer lengths of shanks, consistent with considerations already mentioned, therefore, provide an economical arrangement. Possibility of bank erosion occurring in between the spurs at river stages lower than the design flood stage needs also to be kept in view. For bank protection, spacing equal to 2 to 5 times the projected length of the spur is normally used.

Angle of the spur with direction of approach flow governs scour and formation of separation eddies. When the spur faces upstream, the scour is severe at the nose but scour along shank is much less. Separation eddies are formed both on the upstream and the downstream. Material scoured at the nose forms a shoal by deposition in the eddy portion on the downstream. Such spurs are called deflecting spurs.

With spur facing downstream, the flow hugs the upstream face of the spur without separation and the scour at the nose is less. The downstream eddy and shoal however still

form. The length of bank protected on upstream is shorter but on the downstream, protected length measured from the point of the junction of the spur with the bank is increased. When a battery of spurs is provided, the first spur has to take brunt of the attack and this spur is often oriented facing downstream. Rest of the spurs are made either normal or facing a little upstream. The angle of spurs to the flow direction in case of upstream or downstream facing spurs is ordinarily limited to 20 degrees with respect to normal, upstream inclination being favoured especially in curved reaches.

Spurs are normally made full height with top level above design high flood level providing· an adequate freeboard. When high ground to tie the spur is not available within a reasonable distance, the top level of the spur can be same as the bank level, special care being taken in this case to prevent outflanking by breach near the bank. Downstream face of the spur and the riverbank in the vicinity of the spur need adequate protection. Partial height spurs have been used with success in some countries. Normally, however, full height spurs are preferred.

Length, inclination and spacing of spurs at important locations are finalised preferably by resorting to hydraulic modelling.

5.4.5 Materials of construction

Shank of a spur is constructed of sand core and its exposed faces are protected by stone pitching. The attack at nose is very heavy and hence the nose may be constructed wholly in stone. Alternatively if attack is heavy all over the spur length or if it is easier and not very expensive to construct the spur wholly in stone, such a construction may be preferred. The apron should be in stone of the same size as used for slope protection of guide bunds.

Permeable spurs are constructed by driving wooden bally piles to achieve penetration below riverbed of 6.0 to 9.2m. The ballies are normally 22.5 to 37.5 cm in diameter. Mattress or stone carpet is provided on river bed to prevent scour.

5.4.6 Provisions as contained in IRBM para 811 with regards to Spurs/Groynes

5.4.6.1 A spur/groyne is a structure constructed transverse to the river flow and is projected form the bank into the river.

5.4.6.2 Type of Spurs/Groynes

i) They may be either "Permeable" or "Impermeable". Permeable spurs are constructed by driving wooden bullies or bamboos, filled in with brush wood, with sarkanda mattresses or other suitable material. These are helpful in causing quick siltation due to damping of velocity. They are useful when flood velocities 10 are relatively lower and concentration of suspended sediment load is heavy.They allow water to pass through. Permeable structures are discussed in detail in Para 811(5). Impermeable spurs are made of solid core, constructed of stones or earth and stones with exposed faces protected by pitching. These spurs can withstand severe attack better than permeable spurs.

ii) Spurs may be classified as (a) repelling (deflecting) (b) attracting and (c) normal (sedimenting). Repelling (deflecting) spurs are those which incline upstream at an angle of 60 degree to 70 degree to the river course and deflect the current towards the opposite bank. They cause silting in still water on the upstream pocket. Attra cting spurs incline downstream and make the deep channel flow continuously along their noses. They cause scour just on the downstream side of the head due to turbulence. The river flow is attracted towards the spur. Normal (sedimenting) spurs are those whi ch are built at right angles to the bank to keep the stream in a particular position and promote silting between the spurs. They have practically no effect on the diversion of the current and are mostly used for training of rivers for navigational purposes.

iii) Spurs are also classified as full height spurs and part height spurs. Where top level is higher than HFL, it is called a full height spur.

iv) Spurs are also constructed extending into the stream with a "T" head or hockey stick shaped head, properly

armoured to hold the river at a distance. A series of such spurs/ groynes correctly positioned can hold the river at a position away from the point intended to be protected. The edge of the "T" head should be curved somewhat in the manner of a guide bund to avoid swirls.

5.4.6.3 Location and salient features of a Spurs/Groyne

i) The space between spurs or groynes generally bears a definite ratio to their length. The common practice is to keep the spacing at about 2 to 2.5 times the length so as to effectively protect the bank.

ii) If designed as a full height spur, care should be taken to see that spurs are built sufficiently high so that they are not overtopped and out flanked by the current during high floods. Free board of 1 meter is provided.

iii) The side slopes of spurs are generally 2:1.

iv) The spurs should be anchored on to high ground.

v) The head of the spur is most vulnerable point for scour and should be well protected on slopes by pitching and at toe by an apron designed for scour depth of 2.5 to 2.75 times DLacey at the mole head. For computation of DLacey, Clause 4.6 of 'IRS Code of Practice for the Design of Substructures and Foundations of Bridges' may be referred.

vi) Spurs should never be constructed at a point where severe attack is taking place but at some distance upstream.

vii) Spurs/groynes should be used only in situation where they are absolutely necessary.

viii) The design of spurs may be finalised preferably through hydraulic model studies.

ix) For design and construction of groynes (spurs)/ launching aprons reference may be made to IS:8408-1994 (Planning and Design of Groynes in Alluvial Rivers –

Guidelines) and IRC:89-1997 (Guidelines for Design and Construction of River Training and Control Works for Road Bridges).

5.4.6.4 Maintenance of Spurs/ Groynes

In all cases, satisfactory arrangement should be made for the maintenance of spurs/groynes by providing access to them during all seasons of the year and keeping boulders as reserve. The maintenance procedures specified for guide bunds apply equally to spurs/groynes also.

5.4.6.5 Permeable structures

a) Permeable structures can be used either independently or with the support of other impermeable stone structures or river training and bank protection measures. These structures are easy to construct, use low cost locally available material and require limited skill in construction. These are very handy in anti-erosion works during emergencies in floods. These structures can also be used in areas where good quality stones are costly and/or not available. Thus permeable structures are cost effective alternative to the river training or anti-erosion works with impermeable spurs. Depending upon the purpose to serve, the permeable structures are constructed transverse or parallel to the direction of flow. Permeable structures serve one or more of the following functions:

i) Training the river along a desired course.

ii) Reducing the intensity of flow at the point of river attack.

iii) Creating a slack flow to induce siltation in the vicinity of the permeable structures and in the downstream reach.

iv) Providing protection to the bank by dampening the velocity of flow along the bank.

b) The permeable structures can be classified as follows:

i) According to function served, namely, diverting and dampening, sedimenting.

ii) According to the method and material of construction, namely, bally, bamboo, tree and willow

structures.

iii) According to the conditions encountered, namely, submerged and non-submerged.

iv) According to the type of structure provided, namely, spur type, screen type or dampeners (revetment) type.

c) The permeable structures are made up of different types of smaller units called elements. Many elements, made up of bamboos, ballies, RCC poles etc. are arranged in specific pattern and linked together to form a permeable structure. Different types of elements used for making permeable structures are as following:

i) Porcupines–Porcupines are typically made up of bamboos/ballies, have cubical/prism shaped box at the central portion with their legs extending in all directions. The overall size is 2m to 3m. The central box is filled with stones for stability of individual unit during floods.

ii) Cribs– This is a pyramid type of structure made up of bamboos/ballies with a box at the bottom for holding stones for stability during floods. Size of the box is generally square of size 2m to 2.5m at the bottom. Total height of the structure is 3m to 4m.

iii) Bally frames -Permeable bally structures are made up of main skeleton of large bamboos or ballies. Cross ballies are used for stability of the structure.

iv) Tree branches– Branches of trees or trees of short height are hanged from a wire rope duly weighted with stones and are aligned as a spur projecting into the river. The wire rope is duly anchored on the bank and in the riverbed.

d) The main criteria for the selection of the material are cost and easy/local availability. Standard, commercially available bamboos of girth 20cm to 30cm are used for the porcupines and cribs. Smaller girth of 20cm to 25cm is used for bracings. Standard, commercially available ballies of girth

15cm to 25cm are used for the bally structures. Normally, the larger girth of 20cm to 25cm is used for the main members, whereas, the smaller girth of 15cm to 20cm is used for bracings. Generally, 4 to 5 strands of 4mm GI wire are used for interconnecting porcupines, cribs, and anchor them to the ground. Ballies driven into the ground upto a depth of 2m are treated as anchor. Concrete anchors have an anchor rod of size 32-36mm, well embedded in concrete cube. Wire crate anchors are of size 1.5m x 1.5m x1.5m, made up of thick wires and filled with stones or bricks. A concrete block is casted with bolt and is included in the wire crate anchor. In case of emergencies, tie wires are joined directly to the wires of the crates.

e) In case of shallow water flows and upto maximum depth of flow 3m to 4m, porcupines are used for both spurs and screens. For maximum depths of flow from 4m to 6m, cribs are preferred. For the depths beyond these limits, bally spurs are preferred.

f) Permeable structures commonly used are spurs, dampeners and screens.

i) Spurs are generally made up of 3 to 4 rows of porcupines or 4 to 6 rows of cribs. Schematic sketch of typical permeable spur is shown in Annexure-8/ 6(d). On a straight reach, permeable spurs are normally spaced at 3 to 4 times its length. On a curved channel, depending upon the obliquity of flow, the spurs are normally spaced at 2 to 3 times the length. Projection of the spurs into the river channel is normally 11% to 15% of width of channel. Three spurs are normally provided for a specific reach to be protected. A single permeable spur is generally not found effective. Alignment of spurs is kept pointing towards upstream.

ii) For depth of flow up to 3m, two rows of porcupines are laid along the banks on either side at the toe as dampeners. For more depth, numbers of rows are increased.

iii) Permeable screens are used for choking the secondary channels. 4 to 6 rows of porcupines or 6 to 9 rows

of cribs are normally used in a permeable screen. One screen is normally provided at the entrance of the bypass or secondary channel. The second screen is provided at a distance of 1 to 1.5 times width of the screen and is extended on both the banks for a length one third of the channel width.

g) Due to inherent weakness of the elements, the counter weights are provided in the central box of the porcupines or in the bottom tray of the cribs. Due care is necessary to tie the weights to the main body of the elements. The elements are tied to each other by wire ropes. The tie ropes are duly anchored to the bank and at the nose with the help of suitable anchor or anchor blocks. Intermediate anchors are also provided at an interval of 15m to 20m along the length of the structures on the upstream side.

h) No bed protection is needed for the structures made up of porcupines and cribs. Sinking of these structures into riverbed is a welcome feature, which adds up to the stability during floods resulting in better performance.

5.5 APPROACH BANKS AND MARGINAL EMBANKMENTS

5.5.1 Approach banks

Since country slope is towards the river approach banks are usually required to be provided on either side of the bridge. When the approach cuts off, whole or part of spill discharges, parallel flow is developed along its upstream face, and at the abutments deep scour holes are formed due to concentration of obstructed discharge. In alluvial rivers when bridges are provided with constricted waterway, guide bunds are normally added. The river can then form a single or double loop behind the guide bund. If the guide bund is not sufficiently long, such an embayment can cut into the approach bank. Even if length is adequate to keep the embayment away from the approach bank, parallel flow may still be obtained during floods. Adequate protection is required to be given to the upstream face of approach bank against all such eventualities.

The approach banks need to be made sufficiently high since during floods the water level on upstream side is governed

by the high flood level, which is further raised by the approach velocity. On the downstream side, the water level may be very low. Under these conditions, the approach bank has to withstand high differential head. Rapid draw-down during falling flood can create further, instability in the approach bank. Nature of foundation, material used in forming the approach bank, cross section and protection on side and at toe are important aspects governing stability of an approach bank. Cross section of the bank and surface protection are accordingly designed considering foundation strata, engineering properties of the soil forming the bank and the hydraulic conditions which the bank has to withstand. Necessary precautions are required to be taken, in consonance with prevailing conditions. For instance, in order to discourage formation of rat holes, inverted filter and suitable other means may have to be resorted to.

Alignment of approach has relevance in the context of possible embayment forming at the back of a guide bund and likelihood of active channel developing along the approach bank. Approach bank alignment deviating downstream towards the khadir edge is helpful to reduce the attack under these conditions. When approach banks are considered vulnerable or threatened by river attack, an advance low bank termed as sub bank is sometimes provided to act as the first line of defence.

When river khadir is wide, approaches become longer and water can stagnate behind the guide bunds over long lengths. It is inadvisable to provide culverts or secondary bridge openings with liberal capacity to drain this water. Such openings may encourage the river to develop a direct course through them which is dangerous.

5.5.1.1 Provisions as contained in IRBM para 817 with regards to protection of Approach Banks

1. Approach banks of bridges may be subjected to severe attack under the following conditions:

i) When the HFL at the bridge is very high and there is spill beyond the normal flow channel.

ii) When the stream meets a main river just downstream of the bridge.

iii) In the case of bridges with insufficient water way.

iv) The wave action on the approach bank of bridges situated in a lake/large tank bed may have a detrimental effect.

In all the above cases the pitching of the approach bank up to HFL with sufficient free board is an effective solution. Provision of toe wall and narrow apron in some cases will also be useful.

2. If deep borrow pits are dug near the toe of approach banks, the water flows through these pits and forms a gradually deepening water course which may eventually threaten the safety of the approach bank. In this case it will be useful to put rubble "T" spurs across the flow to reduce the velocity and expedite silting of the course.

3. Whenever the water level on either side of an approach bank is different, there may be seepage of water and to ease the hydraulic gradient, widening of banks, provision of sub banks and toe filters etc may be resorted to.

4. At locations with standing water against the embankment, special watch should be kept when the water level recedes rapidly and when slips are likely to occur.

5.5.2 Marginal Embankments

When railway lines are located on both the banks of a river as in case of the Ganga, the railway banks act as marginal or flood embankments. Since the spill depth can be considerable, height of these banks is usually large. When river is in flood, spills may develop parallel flow along riverside face of the marginal banks. Differential head across these banks can act in both directions depending on the lag in timings and duration 0f floods in the parent river and tributaries.

If marginal embankments are located too near the khadir edge, changes in river meanders may endanger their safety by causing breaches. Alignment and section of marginal embankments have to be properly designed keeping in view all the factors.

Effect of marginal or flood embankments is to restrict the spill discharge and divert it to the main channel. Discharge intensity in the river channel is accordingly increased which improves its sediment transporting capacity. The riverbed is, therefore, normally lowered. Such a lowering is however not reflected in flood levels since confinement for spill flow results in boosting up of water level as is exemplified in case of the Mississippi river. If the river is initially of aggrading type construction of flood embankments may not be effective in neutralising this tendency and the river may continue to aggrade though at a slower rate.

5.5.3 Provisions as contained in IRBM para 812 with regards to Marginal Bunds

Marginal bunds are provided to contain the spread of the river when the river in flood spills over its banks upstream of the bridge site over wide area and likely to spill in the neighbouring water courses or cause other damages. The marginal bund should normally be built well away from the active area of the river. The slope should be well protected by turfing. Where a marginal bund has to be built in the active area of the river, it should be protected with pitching and apron. The earth for the construction of marginal bund should preferably be obtained from the river side. The upper end of the marginal bund should be anchored into high ground well above HFL. Marginal bunds should be inspected every year along with the annual bridge inspection and necessary repairs should be carried out before the onset of monsoon. Cattle crossing and rodent holes across the marginal bund should be specially watched and deficiencies made good.

5.6 BANK REVETMENT

Eroding bank of a river can be protected by constructing spurs or by means of bank revetment. Spurs cause constriction of river width, heavy scour and turbulence at noses, eddies on upstream and downstream side and may involve prohibitive cost. In comparison, bank revetment may be found to be economical and does not normally affect river regime. Bank revetment is, therefore, sometimes found preferable to spurs. Disadvantage of a revetment is that it creates a deep channel along its toe and

thus tends to pin down the channel position which may not be desirable under certain conditions. Secondly the eroding current brushes past the pavement and is not diverted away from the bank as in case of spurs.

Revetment can be of various types. Most common revetment is with stones, either handset or machine placed, riprap or stone in crates. Brick pitching may be laid on edge or pitching may be formed of brickbats in crates or of brick blocks. Alternatively pitching may be formed of soil cement blocks, concrete blocks. concrete slabs or artificial body forms like tetrapods. Articulated concrete mattress is extensively used on the Mississippi river. Pavement formed of interconnected concrete slabs is also popular and commonly used in European countries and the U.S.S.R. Asphalt mat, asphalt carpeting, lumber mattress, bamboo mattress, fascine mattress also provide good surface protection though their life cannot be as long as of stone or concrete pavement. In emergency works gunny bags filled with sand or ballast from track are used when stone is not available. Plastic bags filled with sand and properly sealed are preferred as their life is longer. However several patented systems of erosion control mattresses of synthetic fabric are available and are far too superior. Different types of prefabricated synthetic fabric forms are in vogue which can be laid at site and then filled by pumping mortar. Polyethylene tubes upto 2m diameter size, hydraulically filled with sand after placement at site have also been used successfully [5-15].

Revetment is laid on riverbank after grading the side slope. Eroding banks often acquire quite a steep side slope whereas the natural angle of repose of pitching material may be much smaller. It is necessary that the side slope of the bank be flatter than the angle of repose of material used for pavement. In plan kinks and protrusions in the bank alignment are required to be removed before laying pitching.

Size and thickness of pitching stone is governed by velocity incident on it and the side slope on which it is laid. Allowance for under water work is required to be made suitably. Adequate toe protection is necessary to ensure safety of slope revetment against undermining and slipping due to toe erosion. In this context, possible bed scour is required to be properly

assessed. Stability of pitching depends on provision of well-designed filter below it. All these aspects have been previously dealt with under slope and apron protection for guide bunds.

5.7 CLOSURE BUNDS

Spill channels or bye channels of a river are often required to be closed to obviate possibility of attack on bridge approaches or marginal banks. Such closures can be effectively and economically achieved by means of permeable screens of construction similar to permeable spurs. When such closure work is across the channel carrying substantial discharge, penetration of piles, apron protection, bracings along and across flow direction, stay supports, etc. are required to be designed with adequate factor of safety.

5.7.1 Provisions as contained in IRBM para 813 with regards to Closure Bunds.

Sometimes it may be necessary to entirely block one or more channels of the river in order to prevent the discharge of such channels developing into a main river channel after the construction of the bridge. This is done by providing a closure bund. The bund is designed as an earthen dam. The same is generally constructed at some distance from the Railway line. Special care should be exercised to guard it against its failure. It should be inspected every year after the monsoon and necessary repairs carried out.

5.8 ARTIFICIAL CUT OFFS

Natural cut offs occur in meandering rivers across bottlenecks, especially in hairpin bends, due to progressive bank erosion. In Punjab rivers ratio of bend to chord length when cut off occurred has been roughly found to be 1.7 and in Indus about 2.0.

Cut offs are some times required to be brought about artificially. Sharp bends in rivers hamper navigation and cause shallow bars and oblique crossings to develop. Navigation is immensely improved if such bends are eliminated by making cut offs as successfully done in Mississippi river.

Artificial cut offs across hairpin bends are quite useful as a flood control measure. Consequent upon making a cut off, the river slope on the upstream steepens, velocities increase, bed scours and flood levels are lowered. In the Mississippi lowering of flood levels by 1 to 2 m is achieved by means of cut offs. Increase in velocity, however, results in accelerated bank erosion and consequent change in shape and position of other existing meanders on the upstream and bank protective measures are, therefore, simultaneously warranted.

Cut offs are useful also as a river training measure. Barrages are often constructed in dry and river subsequently diverted to flow through the structure. Such diversions have been found to be aided to a large extent by providing artificial cut offs. When bridges are constructed on braided rivers, discharge distribution in the approach to the bridge may· be very non-uniform. Conditions at the bridge can be markedly improved by effecting proper distribution of the flow by means of suitable cut off across islands. Approaches to pump intakes also at times necessitate making cutoffs to establish direct connection with an active channel to ensure adequate water supplies.

5.8.1 Provisions as contained in IRBM para 814 with regards to Assisted / Artificial Cut- Offs

Sometimes when very heavy meandering develops near bridges and there is a danger of its encroaching too heavily into the still water area or otherwise dangerously approaching the Railway embankment, it becomes necessary to dig a cut-off channel which will ultimately develop and help in the diversion of water through it. To effect economy, a pilot channel cut is usually made when there is low flow in the river and full development of the channel takes place during the flood. This cut-off channel should preferably have (i) at
least three times the river's straight regime slope and (ii) the upstream end should take off from where the bed load of main channel has less than the average amount of coarse material i.e. from the active part of the channel where the velocity is more. The entrance to the pilot cut should be bell shaped to facilitate entry of water. The chord loop ratio should normally be greater than 1 to 5 if a successful channel is to develop. Cut off should be planned with care taking all relevant factors into account.

5.9 PROTECTION FOR SHALLOW PIERS

Bridge piers offer obstruction to flow. Velocity field round them is accordingly affected. On upstream side of the pier, stagnation head is built up, the kinetic head changing to potential head. Water level therefore, rises at this point, along sides, the flow concentrates and velocities increase and water level drops. The resulting pressure and velocity distribution develops a horseshoe shaped vortex which causes maximum scour at the upstream end of the pier. The scour at sides is somewhat less and at the tail end it is least. Depth of scour is governed by scouring action as well as sediment feed into the scour hole.

Scour depth around piers changes with river discharge. During floods, scour depth may be large whereas during low water season it may be small. In bridge design, flood scour is important since piers have to be designed to be sufficiently deep below bed level obtained after flood scour.

Devices have been tried to arrest the pier scour such as rings and projections round the piers to break the pressure gradient responsible for generating scour hole. These methods are, however, found to be not sufficiently effective in the case of major bridges. Adoption of piles or pile clumps for reduction of scour is also not in popular use. The normal approach in designing piers is to sink wells deep enough to ensure safety against worst possible scour. In case of shallow piers of existing bridges, suitable protection is given to ensure safety, as given in Chapter 6.

5.10 TRAINING AND PROECTION WORKS IN HILLY AND SUBMONTANE RIVERS

In hilly reaches of rivers, the current velocities are fast and the training and protection works are, therefore, required to be massive and strong. Either stone crates or concrete blocks of sufficient size and weight are used in such works. Interlacing of crates and blocks may become necessary under exceptional circumstances. Current with high velocity can negotiate only a flat curvature and longer length is required for deceleration. Training works are accordingly required to be more extensive and streamlined.

Landslips are common and hence precautionary measures for efficient drainage and stability of bank slopes are warranted.

For foundation, rocky strata may often become available and hence, foundation difficulties may not pose a problem. Similarly in hilly reaches scour problem may not generally be required to be faced.

In submontane region, the channel pattern in normally braided and training works are required to close several minor and bye channels. Major channel may be prone to shift laterally. Heavy and numerous training and protection works are, therefore, required in this reach. In view of high velocity, design considerations are similar to those in hilly reaches. In addition, aggradation may be associated with tendency for building of an alluvial fan, in which case rate of aggradation is required to be examined. If this rate is fast, possibility of controlling it by means of constriction of channel section or by reduction of sediment load is required to be assessed.

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Chapter 6

DESIGN OF TRAINING AND PROTECTION WORKS

6.1 DESIGN DISCHARGE

6.1.1 Estimation of Design Discharge

Design discharge (Q) is the estimated discharge for the design of the bridge and its appurtenances (accessories). This shall normally be the computed flood with a probable recurrence interval of 50 years (Q_{50}).

6.1.1.1 New Lines & Rebuilding of Bridges

Following methods are available for estimation of design discharge (Q 50) as per Sub structure code Para 4.3

1. From actual data (SSC - Para 4.3.1)

Where stream flow records (yearly peak discharges) are available for the desired recurrence interval or more, design discharge shall be: The computed flood for the desired recurrence interval

2. Statistical methods (SSC - Para 4.3.2)

Where such Stream flow records exist for less than the desired recurrence interval but sufficient for the statistical analysis, design discharge may be: Computed statistically for the desired recurrence interval

3. Unit hydrograph (SSC - Para 4.3.3)

Where records of floods are not available for sufficient length to permit reliable statistical analysis but where Rainfall pattern & intensity records are available for sufficient length of time & Where it is feasible to carry out at least limited observations of rainfall & discharge, Unit hydrograph based on such observations may be developed and design discharge of the desired recurrence interval computed by applying appropriate design storm.

4. Synthetic hydrograph concept & RDSO report RBF-16 (SSC - Para 4.3.4)

Where such observations, as mentioned in Cl. 4.3.3 above, are not possible,

A synthetic unit hydrograph may be developed for medium size catchment (i.e. Area 25 sq. Km or more but less than 2500 sq. Km) by utilising established relationships as mentioned in Flood Estimation Report for respective hydro-meteorological sub zone.

For small size catchment (less than 25 sq. Km), design discharge may be estimated using the techniques described in RDSO report no. RBF-16, titled as "Flood Estimation Methods for Catchments less than 25 km^2 area."

5. Other methods (stage-discharge relationship (SSC - Para 4.3.5)

Where feasible, gauging of the stream may be done to establish the stage– discharge relationships, Discharge at known HFL determined. Otherwise, the discharge may be estimated by slope area method after obtaining flood slope by field observations.

For Indian Rly bridges, the design discharge Q_{50} is generally estimated on the basis of sub structure code para 4.3.4 above i.e. for catchment up to 25 sqkm using RBF-16 and for catchment area 25 sqkm & above and up to 2500 sqkm using flood estimation report (Synthetic Unit Hydrograph concept)

6.1.1.2 Doubling Works & Gauge conversion Projects

G.C. & Doubling Projects (SSC - Para 4.5.7)

i) Where there is no history of past incidents of over flow/washout/ excessive scour etc. during last 50 years: The water way of existing bridge may be retained after taking measures for safety as considered necessary by Chief Engineer In charge.

ii) For locations where there is history of past incidents of over flow/washout/excessive scour: The waterway has to be re-assessed based on the freshly estimated design discharge using clause 4.3.1 to 4.3.4 of sub structure code.

iii) For locations, where existing bridges are less than 50 years old and there is no past history of incidents of over flow/washout/ excessive scour etc.: The water way may be judiciously decided after calculation of design discharge and keeping in view the waterway of existing bridges on adjacent locations on the same river (Para 4.5.7of SSC)

Rebuilding of Bridges (SSC - Para 4.5.8)

For rebuilding of bridge: The waterway shall be determined keeping in view the design discharge as worked out from clause 4.3 of sub structure code.

Gauge conversion projects (SSC - Para 4.5.8)

For strengthening existing bridges by jacketing etc., a reduction in waterway area as per the limits specified below may be allowed by the Chief Bridge Engineer provided that there has been no history of past incidents of overflow/washout/excessive scour etc. and that measures for safety as considered necessary by the Field Engineer and approved by CBE are taken.

Span of Bridge	Reduction in Waterway Area Allowed as % Age of Existing Waterway.
Up to and including 3.05 m	20%
3.05m to 9.12m including	Varying linearly from 20% to 10%
Greater than 9.12m	10%

Further reduction in the area shall be subject to CRS sanction and submission of detailed calculation of waterways etc. Where the clearances are not available, the bridge should be rebuilt.

6.1.2 Design discharge for foundation (Q_f)

To provide for an adequate margin of safety, the foundation and protective works of a bridge should be designed for a flood discharge of higher magnitude than the waterway design discharge (Q_{50}). For this purpose, the waterway design discharge (Q_{50}) may be increased by the percentages given in Table. 6.1.

Table 6.1 : Foundation Design Discharge (Q_f)with Percentage Increase to Waterway Design Discharge (Q_{50})

For rivers having catchment areas at bridge site (Sq. km.)	Percentage increase in waterway design discharge (Q_{50}) to obtain foundation design discharge (Q_f)
Upto 500	30%
More than 500 and upto 5000	30 to 20% (Decreasing with increase in area)
More than 5000 and upto 25000	20 to 10% (Decreasing with increase in area)
More than 25000	less than 10% (using discretion)

6.2 DESIGN OF WATERWAY

When the river flows between high banks and the whole width is actively functioning during high floods, the bridge waterway should practically be equal to the spread between stable banks at the design flood level. This practice may be followed even if such a river slightly overtops the banks during extraordinary floods. These conditions may normally be met with in incised rivers and in upland reaches in river gorges.

If in the above type of rivers, depth of spill is appreciable, waterway should be suitably increased beyond bank to bank width in order to carry spill discharge without causing afflux beyond permissible limit.

In case of alluvial rivers, the bed is scourable. If width at design flood stage in such rivers is more than the width given by Lacey formula

$$P = W = 4.836\ Q^{1/2}$$

It would be desirable to constrict this width at bridge section to achieve economy as well as improve the hydraulic performance. According to Railway Bridge Sub Structure Code,

effective waterway for bridges on alluvial rivers should normally be equal to width given by the Lacey formula.

$$P = 1.811\ CQ^{1/2}$$

wherein C is a coefficient normally having a value of 2.67 but which may be varied from 2.5 to 3.5 to suit nature of bed material and characteristics of flow. Higher value of C is applicable to fast running streams.

Obstruction due to piers is allowed for by adding double the sum. of the weighted mean submerged width of all the piers including footings for wells to arrive at the total width of water way to be provided between ends of the bridge according to Railway practice. [6.7] Similarly one extra span is added to allow for obstruction due to training works in the end spans.

If bank to bank width of an alluvial river is shorter than Lacey width, the bridge should span the entire width but not be extended further to make the waterway equal to Lacey width.

Even if the river is alluvial, but the flood is flashy, time may not be sufficient to cause bed scour, constriction on basis of Lacey width is then not permissible. In such cases maximum depth of scour should be ascertained from field observations and waterway determined by Velocity- Area method. If river data of velocities at flood stages is not available, a suitable velocity formula may be employed for this purpose. Progressive scour-afflux computations may also be made for the design hydrograph as explained in Chapter 4 and results used in finalising the design waterway.

For rivers in submontane region, the slopes are steep, velocities high, bed material ranging from heavy boulders to gravel and floods often of flashy nature. The constriction in such rivers should be governed largely by the configuration of active channel or channels, the cost of diversion of channels and the cost of guide bunds which can be much longer needing heavier protection than in case of guide bunds in sandy bedded rivers.

In tidal reaches of rivers, the regime is predominantly governed by tidal characteristics. Constriction is often likely to

affect the prevailing tidal pattern and cause deterioration by silting. The possibility of constriction of a tidal waterway should, therefore, be viewed from all angles including tidal regime, navigation requirements, effect of salinity, etc.

6.3 GUIDE BUNDS

6.3.1 Form in Plan

Guide bund may be given an elliptical shape as in Figure 6.1 since separation of flow can be avoided better with such an alignment. Alternatively straight parallel guide bunds can also be adopted if preferable on account of any reasons. Straight parallel guide bunds may be provided with heads of sufficiently flat curvature or of composite curve, to avoid separation of flow from the guide bund head and shank.

In case of submontane rivers and when there are site constraints, deviation from elliptical shape may be warranted. Guide bunds shapes conforming to channel configuration and site conditions with wide splay and along length may then be necessary.

6.3.2 Length of guide bund

For encouraging axial flow through the bridge, upstream length of guide bund may be kept 1.0 to 1.5 times the bridge length and downstream length 0.25 to 0.40 times the bridge length. Length necessary for keeping worst possible embayment behind the guide bund sufficiently away from the approach bank is also required to be ascertained by fitting of sharpest loop as indicated in Figure 5.2 in Chapter 5. If field data of acute bends in river when cut offs occurred is not available, It may be assumed that radius of such a bend can be equal to 0.4 times the radius of average bends if maximum river discharge is up to 5660 m^3/s and equal to 0.5 times the radius of average bends if the maximum discharge is more than 5660 m^3/s. In case of braided rivers, the length determined on basis of both the considerations of axial flow through bridge and keeping away the worst loop may be found to be insufficient for protection of long approaches. Additional protection works like spurs, revetments etc. may then become necessary.

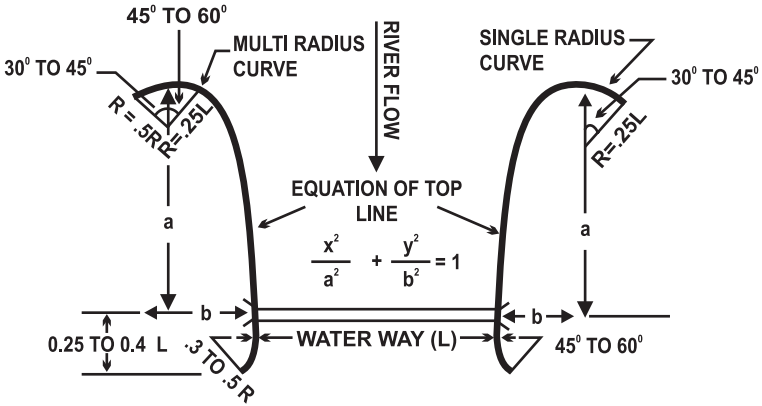


Fig. 6.1 : Elliptical shape of guide bunds

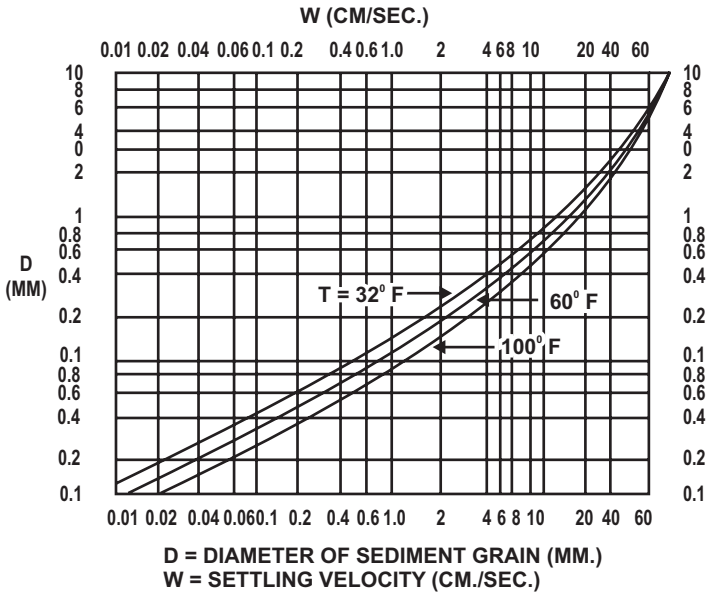


Fig. 6.2 : Settling velocity of sediment grains of different sizes

On the other hand if constriction is small on one or both sides and exposed length of approach is accordingly short, guide bund of length smaller than the length of the bridge may be found adequate to ensure safety of the approach from worst loop and also to guide flow smoothly without separation, obliquity and with optimum uniform discharge distribution across the bridge. Shorter length than bridge length for the guide bund is considered permissible under these conditions.

6.3.3 Curved heads

In case of rivers carrying high discharges of the order of 28300 m^3 *Is* and above, Gale's recommendations are more conservative than Spring's. For rivers with discharge of the order of 14150 m^3/*s,* radius of curve according to different authors is nearly same. For discharges smaller than 14150 m^3/s radius can possibly be shorter than suggested by Spring. In the absence of definitive overall criteria, judgement needs to be used in selecting suitable radius or radii when the curve is composite. In case of major bridge projects, aid of hydraulic model studies may, in addition, be sought in finalising the curve.

6.3.4 Cross section and protection

(i) Top Width

Sufficient top width may be provided for the guide bund to permit easy transport of construction materials. It should be in any case not less than 6 m.

(ii) Top Level

Top level of guide bund should allow a minimum free board of 1 m above affluxed high flood level corresponding to foundation design discharge. On the rear side of the guide bund, water level rises on account of ponding by an amount equal to velocity head. When heavy wave action is expected, free board should be suitably increased.

(iii) Side Slopes

The side slopes depend on the nature of river bed material of which guide bund core is formed and the height. Ordinarily side slope of 2: 1 on riverside and 2:1 to 3:1 on the other side may be found adequate.

(iv) Material

Core of the guide bund section may be constructed with non-cohesive bed material such as sand and gravel, obtained from riverside and not from back side. Clay should not be used for core construction.

(v) Size of pitching stone

'One man stone' of 30 cm to 40 cm size weighing from 40 to 70 kg which can be handled by a worker may be used. Such a stone can withstand an average velocity up to about 3.5 *m/s.* When average velocity is smaller, upto about 2.5 *m/s,* burnt brick pitching on edge laid by hand may suffice. For average velocity higher than *3.5 m/s,* crated stone or concrete blocks may be used, their size or weight being in accordance with the relationship

$$V = 5.7\ D^{1/2}$$

and $$W = 0.04\ V^{6}$$

wherein V is the average velocity in *m/s, D* is the diameter in m of spherical crate of equivalent volume and W is weight in kg of a crate of non-spherical shape. Suitable allowances are required to be made to this size and weight as explained further. Crates may be made out of 8 gauge G.I. wire of 4.064 mm dia with double knots and closely knit to prevent stones slipping out.

Incident velocity on the stone or crate is always less than the average velocity on the vertical, the relation between the two being [6.3]

$$V_{bottom} = \frac{0.71\ V_{average}}{0.68\ \text{Log}\ y/D_{50} + 0.71}$$

wherein y is the flow depth in m and D_{50} is also in m. Adoption of average velocity in place of bottom velocity introduces a certain factor of safety.

The V-D relationship is meant for stones with and without crates having weight of 2645 kg/m^3 or specific gravity of 2.65. If specific gravity of stones is different, the size can be corrected using Creager equation.

$$K_w = \frac{102.5\ K}{0.672\ w - 62.5}$$

wherein K_w is the size in m with weight of stone of w (kg/m^3) and K is the stone size in m with weight of 2645 kg/m^3.

When stone or a crate is placed on a side slope, its stability is reduced. Relation between permissible velocity for stability on side slope to that on horizontal bottom is given by [6.3]

$$\frac{\text{Permissible } V_{bottom} \text{ on side slope}}{\text{Permissible } V_{bottom} \text{ on horizontal bottom}} = \left[1 - \frac{\text{Sin}^2\varphi}{\text{Sin}^2\theta}\right]^{1/4} = x$$

wherein ϕ is the angle of side slope and θ is the angle of repose of the stone. Values of x for various side slopes assuming θ of 40° are given below.

Side Slope	1 ½ : 1	2 : 1	3 : 1	4 : 1
Value of x	0.71	0.855	0.935	0.953

Permissible velocity V on horizontal bottom is required to be multiplied by the factor x to find permissible velocity on side slope for given size of stone or crate. Alternatively for given velocity, if size of stone or crate on side slope is to be determined, V is required to be multiplied by 1/x. The relationship $V = 5.7D^{1/2}$ is meant for use when side slope is 2 horizontal to 1 vertical. For side slopes other than 2:1 suitable multiplying factors have to be adopted. Additional allowances in stone or crate size to account for severity of attack, obliquity of flow, wave action, higher level of turbulence etc. should be made using Table 5.8 of Chapter 5 for guide.

(vi) Thickness of pitching

Pitching constructed of one man stone weighting 40 to 70 kg is used in Indian practice and no further gradation is allowed. Stones of this size have been found to be stable in velocities up to about 3.5 m/s inclusive of all factors such as severity of attack, slope effect etc. After considering all available data, Inglis recommended that the thickness T of pitching should be worked out according to formula $T = 0.06\ Q^{1/3}$, wherein Q is discharge in m^3/s and T thickness of pitching in m. This formula is therefore, proposed for adoption out of various formulae referred in Chapter 5.

Thickness is increased by 25 per cent at head to take care of severe attack due to exposure and 25 per cent all over when pitching is dropped through deep water below low water level. Thickness can be reduced by 150 to 225 mm by using quarry refuse or burnt bricks as filter. The final thickness so arrived at should not however exceed the upper limit of 1.3 m.

In following theoretical practice, the stones to be used in pitching should be well graded is as indicated below.

Size of Stone	Percentage of total weight smaller than given size
3 k	100
2 k	80
1 k	50
0.1 k	not to exceed 10

in which k is the diameter of stone that will have the weight same as 50 per cent size of stones D_{50} arrived at applying the formula

$$V = 5.7\ D^{1/2}$$

$$D_{50} = 0.031\ V^2$$

$$W = 0.04\ V^6$$

wherein W is in kg/m^3 and V and D_{50} are in m.

Thickness of such graded stone protection should be $2D_{50\ max}$ or $1.5D_{100\ max}$ whichever results in greater thickness. Thickness should not be less than 1.5 D_{50} and not less than 31 cm.

The graded stone pitching is placed on a filter of proper design. Size, gradation and thickness of the filter material should be such that bank material is not sucked out through voids. Standard specifications generally require –

$$\frac{D_{15} \text{ of riprap}}{D_{85} \text{ of bank material}} < 5 < \frac{D_{15} \text{ of riprap}}{D_{15} \text{ of bank material}} < 40$$

The thickness of filter blanket ranges from 15 to 23 cm for a single layer and from 10 to 20 cm for individual layers of a multilayer blanket.

Synthetic mesh nets of woven fabric provide a good filtering medium and can replace granular filters especially when laying filter under water becomes difficult.

6.3.5 Launching Apron

(i) Size of stone

Size of apron material may be kept same as for side slope protection provided thickness is according to Indian practice advocated by Inglis and others. Otherwise size can be as recommended by the ISI viz

$$W = 0.10\ V^6$$

(ii) Thickness

Thickness of apron when launched should be 25 to 50 percent more than the thickness of pitching on slope.

(iii) Slope of launched face

Launching slope may be assumed as 2 horizontal to 1 vertical.

(iv) Quantity of apron stone

Quantity should be sufficient to cover the scoured face down to maximum depth of scour obtained using scour factor c of 2.5 at upstream curved head and 1.5 for the rest of the length in Lacey formula

$$D = c \times 1.34 \left(\frac{q^2}{f}\right)^{1/3}$$

wherein D is depth of flow below H.F.L. in m, q is discharge intensity in m^3/s per metre width within guide bunds on bridge section, f is the silt factor and c the scour factor. It has been observed that the above Lacey formula does not give a good fit for data of some of the rivers. As an alternative, the Laursen formula for guide bund contraction can therefore be applied in the following form to obtain average depth of flow D within guide bunds.

$$\frac{\text{Depth within guide bunds}}{\text{Depth of unconstricted discharge}} = \left[\frac{\text{Width of unconstricted channel excluding spill portion}}{\text{Width within guide bunds}}\right] X \left[\frac{\text{Discharge of channel plus spills}}{\text{Discharge of channel}}\right]^{2/3}$$

wherein

$$X = 0.59 \text{ when} \left(\frac{\sqrt{gDS}}{w}\right) = 1/2$$

$$X = 0.64 \text{ when} \left(\frac{\sqrt{gDS}}{w}\right) = 1$$

$$X = 0.69 \text{ when} \left(\frac{\sqrt{gDS}}{w}\right) = 2$$

In $\sqrt{gDS}/w$, g is gravitational acceleration in m/s/s, D is depth of flow in m, S is river slope and w is velocity of fall of sediment particles of size corresponding to mean diameter of

bed material in still water, which can be estimated from curves in Fig. 6.2.

The scour depth can be arrived at by using scour factors mentioned before. Bigger of the two scour depths, obtained using Lacey formula and Laursen procedure should be adopted for apron computations. At the curved head, launching surface becomes fan or cone shaped with length of sloping face equal to 2.3 times the depth of scour below bed level. The length of fanned out surface is equal to $x\pi r$ where r is radius of cone at half the depth of scour below bed level and x is the sweep angle in degrees/180 as shown in figure 5.7 of Chapter 5. The quantity required for covering the fanned out surface should be provided over the entire sweep angle.

(v) Laying of apron

Width of apron laid on river bed should be 1.5 D_1 where D_1 is estimated scour depth in metres below river bed. Thickness of apron as laid can be worked out as quantity of apron stone/ width of apron laid and works out to 1.5 T_1 where T_1 is the thickness of apron after launching. The thickness at the head should be increased to 2.25 T_1 and checked up to verify whether the quantity is adequate for fanning out. If not, the thickness should be increased further as required. Apron should be turned round the noses on upstream and downstream sides. Fig. 6.3 illustrates important design features.

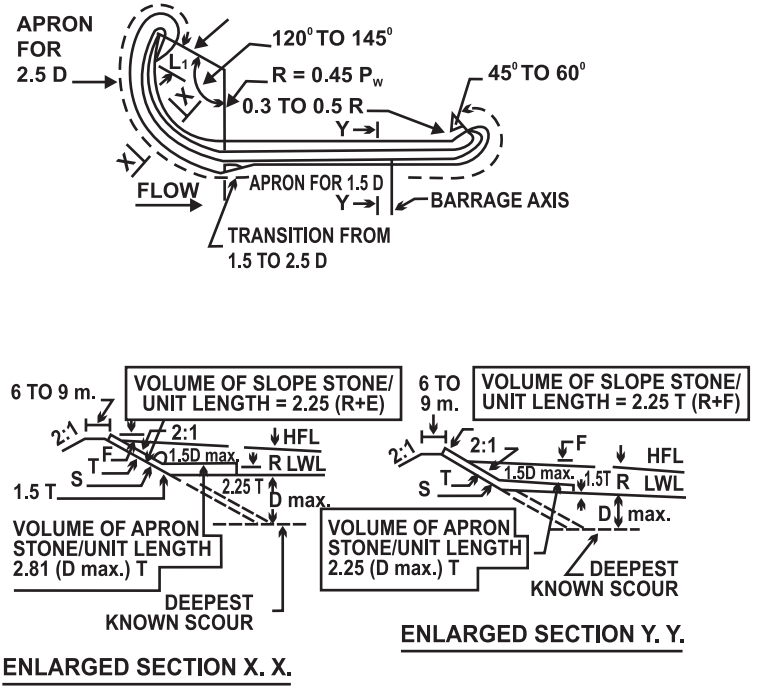
6.4 SPURS OR GROYNES

6.4.1 Types of Spur

As explained in 5.4.2 solid spurs are generally used for deflecting attack of flood flow whereas permeable spurs are provided primarily for bank protection when flood velocities are relatively lower and flood flow carries heavy concentration of suspended sediment.

6.4.2 Location

Any reach undergoing or threatened by erosion should be determined by field inspection. Previous surveys of bank line,



LEGEND :

F = FREE BOARD, R = RISE OF FLOOD ABOVE LOW WATER LEVEL,
D max. = DEPTH OF SCOUR FOR CALCULATION OF APRON STONE,
T = THICKNESS OF SLOPE STONE, S = THICKNESS OF SOLING (FILTER).

Fig. 6.3 : Apron design features

if available, should be superimposed to demarcate the eroding reach. In bend flow, more vulnerable portion of bank to erosion is generally at the downstream end of the bend.

6.4.3 Length, inclination, height and spacing for impermeable spurs

These details should be fixed following criteria given in Chapter 5.

6.4.4 Cross section and protection of impermeable spurs

The following guidelines are generally in accordance with Indian Standards 8408-1976.

Top width is generally kept at 3 to 6 m to allow vehicular transport for construction materials.

Free board of 1.0 to 1.5 m is allowed on upstream side above anticipated high flood level.

If shank is constructed of sand, side slope can be 2: 1, If stone is used instead of sand side slope can be steeper at 1 ¼ :1 or 1 ½ :1. Nose slope should, however, be 2:1 irrespective of whether it is constructed in sand or stone. It is preferable to construct nose wholly in stone.

Stone pitching should be provided on all exposed slopes of the spur. Size of stone on sloping face should withstand the estimated flood velocity. Ordinarily stone of 40 to 70 kg weight can withstand flow with average velocity up to 3.5 m/s on side slope of 2:1. For higher velocities, stone crates or concrete blocks have to be used. In concrete blocks of required size, pattern grouting may be done which helps to localise damage and adds stability. In general side slope protection should be on the same lines as in case of guide bunds.

The thickness of pitching should be designed in the same way as in the case of guide bunds. The thickness can be reduced from nose backwards along the length of the shank as given in Table 6.2.

Table 6.2

Location	Thickness of pitching T metres or fraction of T	Length of spur to be provided with pitching of thickness in Col.2
1	2	3
Nose	T	Whole
Upstream side of shank	T	First 31 to 46 m or scour hole length whichever is larger.
	2/3 T	Next 31 to 46 m
	Nominal 0.3 m	Rest
Downstream side of shank	2/3 T	First 31 to 61 m
	Nominal or Nil	Rest depending on action of return flow

Graded filter 20 to 30 cm thick satisfying standard design criteria should be provided at the nose, and over the shank. When thickness of pitching is kept nominal, thickness of filter can be reduced to 15 cm or it can be omitted considering likelihood of parallel flow and its action.

Apron of standard design as in case of guide bunds should be provided for protection of scoured face down the entire depth of scour hole.

Size of apron stone or crates or concrete blocks should be the same as in case of guide bund. Slope of launching apron may b assumed to be 2: 1.

Scour depth may be found according to the norms given Indian Standard 8408[6.4] or after experiments conducted by Mustaq Ahmed [6.5] as in Table 6.3.

Table 6.3 : Scour Factors for Apron Design in case of Spurs

(A) According to Indian Standard Specifications

Sr.No.	Location	Scour Factor
(i)	Nose	2.0 to 2.5
(ii)	Transition from nose to shank and first 30 m to 61 m on upstream	1.5
(iii)	Next 31 m to 61 m on upstream	1.0
(iv)	Transition from nose to shank and first 15 m to 30 m on downstream	1.0

(B) On Basis of Experiments conducted by Mushtaq Ahmed

Sr.No.	Location	Scour Factor
(i)	**Nose**	
	For spur facing D/S at 30° to bank line	1.5
	For spur facing D/S at 60° to bank line	2.0
	For spur at 90° to bank line	2.5
	For spur facing U/S at 30° to bank line	3.0
(ii)	**Shank**	
	For spur facing D/S at 30° to bank line On U/S face over length 0.85 L On D/S face over length 0.30 L	 1.0 1.0
	For spur facing D/S at 60° to bank line On U/S face over length 0.80 L On D/S face over length 0.40 L	 1.0 1.0
	For spur at 90° to bank line On U/S face over length 0.70 L On D/S face over length 0.50 L	 1.0 1.0
	For spur facing U/S at 30° to bank line On U/S face over length 0.60 L On D/S face over length 0.72 L	 1.0 1.0

Note :-

(a) Scour factor is defined as scoured depth below H.F.L. divided by Lacey depth below H.F.L.

(b) L is the projected length of an inclined spur given by actual length x sine of angle with respect to bank line.

(c) Out of the apron designs worked out according to A and B above, heavier of the two may be adopted.

Practice followed in this respect in the U.S.A. is given in Table 6.4 for comparison.

Table 6.4 : Formulae in Vogue in the U.S.A. for Prediction of Scour at the Noses of Spurs

S.No.	Author	Formula
1.	Laursen[6.9]	$\frac{L}{D} = 2.75\frac{d_s}{D}\left[\left(\frac{1}{11.5}\cdot\frac{d_s}{D}+1^{1.7}\right)-1\right]$
2.	Liu[6.14]	$\frac{d_s}{D} = 1.1\left[\frac{L}{D}\right]^{0.4} F_r^{0.33}$ Recommended by Richardson[6.15] $\frac{L}{D} < 25$
3.	For spurs in the Missis-sippi river with large L/D ratio	$\frac{d_s}{D} = 4.0F_r^{0.33}$ Recommended by Richardson[6.15] for $\frac{L}{D} > 25$

For arriving at scour depth with spurs making different angles with flow direction, multiplying factors are used.

Note :- In the above formulae :

d_S is equilibrium scour depth below mean bed level, D is upstream depth flow, F_r is upstream Froude Number $\left(\frac{V}{\sqrt{gD}}\right)$ and L is effective or projected spur length.

Thickness of apron should be 25 to 50 percent more than thickness of pitching on slope.

Width of launching apron should be equal to 1.5 times the depth of scour below river bed. At the nose, additional quantity of stone should be provided to allow fanning out along the cone face to scour level as in case of upstream head of a guide bund.

A typical design of spur is illustrated in Fig. 6.4.

6.4.5 Permeable Spurs

A permeable spur may be constructed by driving two or more rows of bally piles with sufficient penetration below riverbed level to provide adequate grip during floods. Cross braces and longitudinal runners or waling pieces may be provided to strengthen the structure. Stays may be added on either ends so that differential pressure and flood attack can be withstood. The structure may be filled with brush wood and weighed down by stones. For more details on permeable structures, Chapter 7 may be referred.

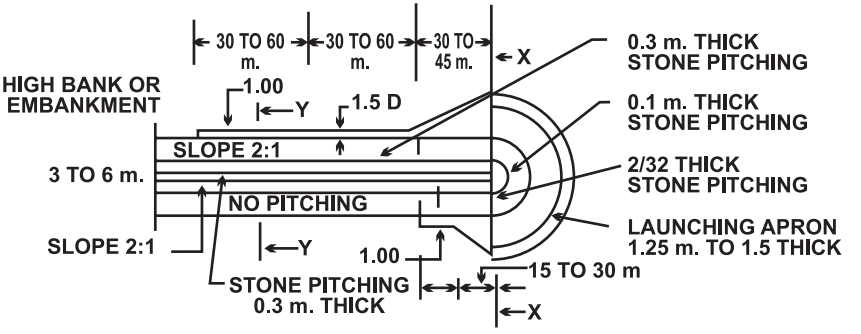
6.5 APPROACH BANKS AND MARGINAL EMBANKMENTS

6.5.1 Approach Banks [6.5]

When approach banks are of medium height upto 3 m, the material may be homogeneous. For bigger heights zoned section with impervious core and pervious outer sections may be desirable.

Foundation should provide a stable support for overlaid bank and should resist harmful percolation. Unconsolidated material in foundation is liable to settlement and percolation of water. Cut off trenches backfilled with rolled impervious material, sheet pile cut off, grouting, clay blanket on upstream side and inverted filter on downstream side are helpful in minimising harmful effect of percolation.

Hydraulic gradient should be well within the bank with



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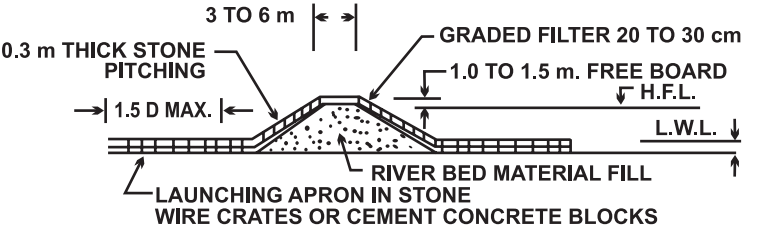
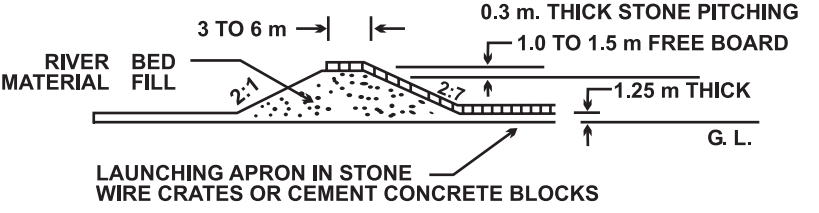


SECTION - XX



SECTION - YY

Fig. 6.4 : Design of impermeable spur

adequate cover. As an approximation it may be assumed as a straight line with slopes of 1: 1 in impervious clay varying to 1: 12 in sandy soil. For ordinary clay the gradient is about 1 in 4. The cover over hydraulic gradient line should be 0.9 to 1.2 m along slope and about 0.6 m along ground.

The approach bank section should be stable for conditions of saturation and sudden drawdown. On the downstream side worst condition for stability can be when there is H.F.L. on upstream side and much lower water level on downstream face. Under all these conditions a factor of safety of 1.3 to 1.5 should at least be ensured.

Top width is guided by consideration of locating railway line or road way.

Top level for approach bank should be worked out by adding to the design high flood level allowances for afflux, rise in water level from bank to guide bund head due to slope, velocity head, wave wash and free board. For upstream slope, the angle should be flatter than the underwater angle of repose of the material. The steepest slope for good soil can be 2:1 for low embankment of height up to 3 m and 3:1 for high embankments. If the material is sandy, the upstream slope may be as much as 4:1.

Downstream slope should be guided by hydraulic gradient and cover. If on account of any reason, removal of material due to piping is noticed, addition of inverted filter and stone pitching may be found to be useful to check erosion.

Side slopes and cross section of the approach bank should also be designed, especially in case of high embankments, on stability considerations based on engineering properties of the material forming the bank and foundation and hydraulic conditions.

Side slope protection should be provided against parallel flow on the same lines as in case of guide bunds. On the downstream side of approach banks, turfing may be provided for protection against rain and wind. Longitudinal and cross drains may be considered if rainfall is heavy and embankment high.

When bridge approaches obstruct spill discharge and guide bunds are not provided, deep scour holes can be formed at abutments. Estimation of scour depth and protection at the abutments for slope and for toe in the form of apron may be similar to that in case of spurs.

6.5.2 Marginal Embankments

Design aspects of marginal embankments are similar to those of approach banks. The aspects needing special attention are dealt with below.

Distance of marginal embankment from the river bank should neither be too short nor too long. If the embankment is too near the river, velocity along the bank can become high. Normally this velocity is not allowed to be more than 0.9 to 1.2 *m/s.* On the other hand if the embankment is far too away, the very purpose of providing the embankment can be defeated. Current Indian practice is to roughly maintain a distance of 3 times the width given by Lacey formula in between the flood embankments on either banks of the river, though this thumb rule may not be applicable in case of all alluvial rivers. The flood embankment is carried on a ridge or high bank. In alluvial rivers a ridge or high ground is often formed near the bank line due to deposition of suspended sediment when river overflows the bank. Protrusions and sudden changes in alignment forming kinks are as far as possible avoided. Structures like bridges and barrages obstruct free downstream movement of meanders. Immediately upstream of such structures, the meander belt, therefore, widens and becomes almost double. While fixing location of marginal banks, this field experience needs to be kept in view.

Height of embankments may be designed by allowing sufficient free board above design high flood level obtained by routing design flood hydrograph.

Since velocities along river side slope of the marginal banks are expected to be low, no slope protection may ordinarily be required and turfing on both side slopes may suffice for protection against rain and wind. In special circumstances, if river attack warrants slope protection, it can be similar to that suggested in case of guide bunds.

6.6 BANK REVETMENT

6.6.1 Side Slope

Angle of bank should be flatter than angle of repose of pitching material. Angle of repose of sand is 2:1 while that of stone is about 1:1. Angle of river bank formed of cohesive soil can however be steeper than 1:1 in which case it is required to be graded and made flatter than 1: 1. Flatter the side slope with respect to angle of repose of pavement material, the size and weight of pitching stone can be reduced.

6.6.2 Velocity in the eroding bend along concave bank

If the average velocity in the river in a straight reach is known but not the velocity along eroding concave bank in the bend, it can be estimated using the relationship (6.3)

$$\frac{T_{bend}}{T_{straight}} = \frac{V^2_{bend}}{V^2_{straight}} = 3.05\left[\frac{r}{w}\right]^{0.5}$$

wherein T_{bend} is maximum boundary shear in kg/cm^2 as affected by bend, $T_{straight}$ is average boundary shear in kg/cm^2 in a straight reach, r is center line radius of bend, and w is water surface width in m at upstream end of the bend.

If average velocity in the bend section is known, corresponding velocity along eroding bank can be roughly estimated also on basis of flow depths using the expression

$$\frac{\text{V at eroding bank}}{V_{mean}\text{ of cross section}} = \left[\frac{\text{Flow depth at eroding bank}}{\text{Average flow depth of the cross section}}\right]^{2/3}$$

When measured flow depth along eroding bank is not available, the following guide lines given by Lacey(6.6) for estimation of the depth may be found useful.

Pattern of channel	Increase in flow depth below High Flood Level
i) Straight reach	…1.25 x normal flood depth
ii) Moderate Bend	… 1.50 x normal flood depth
iii) Severe bend	… 1.75 x normal flood depth
iv) Right angled bend	… 2.0 x normal flood depth

6.6.3 Side slope and apron protection

Design of protection for side slope and of apron may be on similar lines as for guide bunds.

6.7 ARTIFICIAL CUT OFFS

Alignment of a cut off is chosen so that approach and exit ends remain more or less tangential to the river course. A cut off can be made either by open excavation or by means of a dredger. Since considerable amount of work is usually involved below low water level making a cut becomes easier with a dredger. Normally only a small pilot cut of about 10 per cent capacity of the channel discharge is made where the river cut off is desired. The river is relied upon to open out the pilot cut and develop it once the water starts flowing. When arc to chord ratio is about 5 or more, development of a cut off may be expected to be rapid. A deep and narrow cut may be preferred to a shallow and wide section, since side erosion is easier and faster than bed scour. Velocities in deeper cut are also more.

In designing an artificial pilot cut, its progressive development and final adjustment of the river are required to be visualised. When cut off is made across the neck of the bend, length is shortened so that with the available fall, slope steepens increasing the velocities. These increased velocities open out the cut rapidly and the river gets diverted on the new cut off channel. With increased velocities, bed scour and degradation occurs upstream of the cut off. The scoured material is deposited on the downstream side and finally equilibrium bed and water surface slopes are established which can be a little steeper than

the original. Thus in effect river attains a somewhat steeper slope and generates higher velocities. Bank protection is, therefore, usually needed at and in the vicinity of a cut off to prevent side erosion. Otherwise new bends may form in the wake of a cut off as was experienced in the Mississippi River[6.7]. Superimposition of the expected longitudinal section with cut off over the existing section permits a rough assessment of possible degradation and aggradation. Such an attempt should be made to estimate the final bed and water levels and lengths likely to be affected by the cut off.

6.8 BRIDGE PIER PROTECTION

6.8.1 Deep piers

(i) Scour depth and grip length

Present practice is to sink bridge piers sufficiently deep to provide a safe grip length even after full scour occurs around them at the foundation design flood stage.

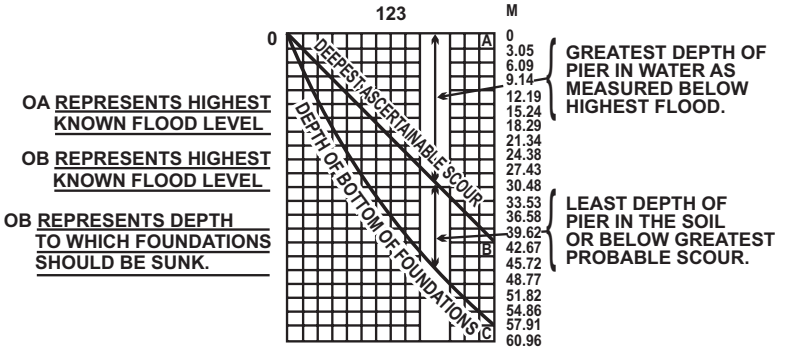
Spring suggested minimum depth of pier foundations based on anticipated scour as shown in Fig. 6.5.

According to this figure the grip below scoured bed level is half the height from scoured bed level to H. F. L. for 30 m height, less than half the height for bigger depth and more than half for lesser depth of flow.

The provision in IRS and IRC codes is that minimum grip should be 2/3 the flow depth in the bridge or 1/4 the total height of pier from bottom to design H. F. L. or 1/3 the flow depth below design H. F. L. after pier scour has occurred. When this grip is ensured by sinking deep wells, no protection against scour is required to be given.

Estimation of scour depth is thus an important aspect of pier design. According to Indian Railway practice based on studies made by Inglis [6.7] pier scour is assumed to be double the normal depth of flow given by Lacey formula

$$D = 0.473\,(Q/f)^{1/3}$$



NOTE :

THE DIAGRAM APPLIES ONLY TO SANDY BOTTOM. IF THE RIVER BED IS SOFT SLUSH A GREATER DEPTH IS NECESSARY. PIERS ARE ALWAYS PRESUMED TO HAVE ENOUGH STONE AROUND THEM TO PREVENT LOCAL PIER-FORMED SWIRLS FROM SCOURING POT-HOLES AT PIER BASE.

Fig. 6.5 : Depth of bridge piers according to spring

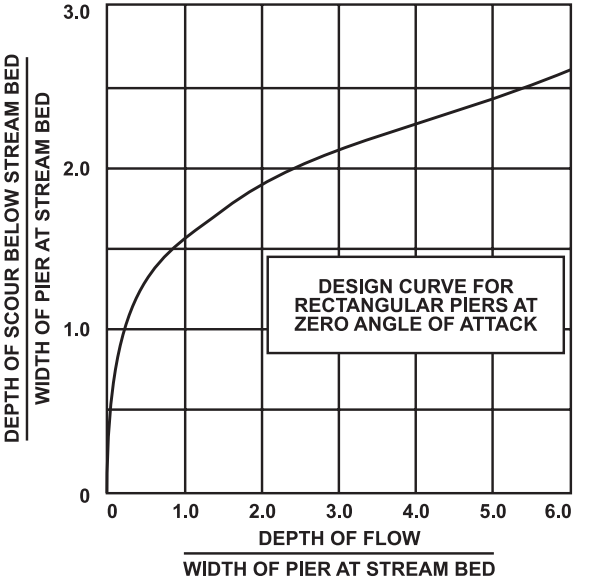


Fig. 6.6 : Relation between pier size and scour depth given by Laursen

The pier scour in IRC code is assumed to be double the normal depth of flow given by Lacey formula

$$D = 1.34\ (q^2/f)^{1/3}$$

When bridge is provided with constricted waterway, bigger of the two scour estimates should be used in design of pier foundations. In the above two Lacey formulae, q is the intensity of discharge in m^3/s per meter width within abutments, D is depth of flow in m nearly equal to hydraulic mean radius in case of wide rivers, Q is the design discharge in m^3/sec and f the silt factor given as 1.76 $m^{1/2}$, m being the weighted mean diameter in mm of the bed material. The weighted mean diameter is obtained by giving weightage with respect to percentage fractions of difference sizes.

If m has nearly the same value over the entire scour depth, working out silt factor presents no difficulty. It is however possible that size of bed material may change at various depths from riverbed to scoured bed level. Silt factors are then worked out for each strata having nearly uniform sized material and scour depth determined adopting each f value starting from rivers bed downwards. If size of bed material is coarser at lower levels, final scour depth will work out to be less than given corresponding to riverbed material. If material happens to be finer at lower depths, the scour depth will work out to be more.

Procedure to be followed in working out scour depth when bed material comprises different sizes at various elevations is given below:

Estimation of Scour Level in River Bed with Varying Sizes of Bed Material at Different Depths.

Design Data

(i)	Foundation design discharge	=	1500 cumecs
(ii)	Corresponding H.F.L.	=	107 m
(iii)	River bed level at bridge site	=	100 m
(iv)	Weighted mean diameter of bed material at different depths.		

Alternative (A) –

From RL 100 m to 98 m – size of bed material 0.32 mm

From RL 98 m to 97.2 m – 0.50 mm

From RL 97.2 m to 90 m – 0.70 mm

Alternative (B) –

From RL 100 m to 98 m – 0.50 mm

From RL 98 m to 96.2 m – 0.32 mm

From RL 96.2 m to 90.0 m – 0.20 mm

It is required to estimate the scoured bed level at a bridge pier adopting scour factor 2 in the Lacey formula

$$D = 0.473\ (Q/f)^{1/3}$$

The stepwise procedure is. given in Table A for alternative (A) and Table B for alternative (B) respectively.

Estimation of scour depth using Lacey formula is an empirical procedure which ignores size of obstruction. No additional allowance for non-uniform discharge distribution, flood scour, bed form etc. is normally made.

Laursen developed a semi-theoretical approach for estimation of pier scour which accounted for the size of the pier. By using Manning flow formula and his own sediment transport function, Laursen first obtained scour depth ds below river bed in case of a long contraction. The scour at abutment was then assumed to be r times that in a long contraction. Value of r was found to be 11.5 by model tests. To obtain pier scour, width of a rectangular pier b (or diameter in case of a circular pier) was considered equal to twice the effective length of the abutment. The Laursen relationship so obtained was the following shown graphically in Fig. 6.6.

Table 'A' : Scour level computations for bed material sizes given in Alternative (A)

S. No.	Elevation	Value of 'm'	Value of 'f' $f = 1.76m^{1/2}$	D Lacey $D=0.473 \left(\frac{Q}{f}\right)^{1/3}$	Scour Level HFL-2D Lacey	Whether estimated scour level is higher than, equal to or lower than level corresponding to 'm'	Remarks	Estimated scour level to be finally adopted
1	2	3	4	5	6	7	8	9
1.	From 100 to 98 m	0.32 mm	1.00	5.45 m	107.00 - 10.90 = 96.10 m	Lower	Scour level will be below RL 98 m	
2.	From 98 m to 97.2 m	0.50 mm	1.25	5.00 m	107.00 - 10.00 = 97.00 m	Nearly equal	Scour will continue to RL 97.2 m	Scoured level will be RL 97.2 m
3.	From 97.2 m to 90 m	0.70 mm	1.47	4.77 m	107.00 - 9.54 = 97.46 m	Higher	Scour will not proceed below RL 97.2 m	

Table 'B' : Scour level computations for bed material sizes given in Alternative (B)

1	2	3	4	5	6	7	8	9
1.	From 100 to 98 m	0.50 mm	1.25	5.00 m	107.00 - 10.00 = 97.00 m	Lower	Scour level will be lower than RL 98.00 m	
2.	From 98 m to 96.2 m	0.32 mm	1.00	5.45 m	107.00 - 10.90 = 96.10 m	Nearly equal	Scour will continue to RL 96.10 m	Scoured level will be RL 96.00 m
3.	From 96.2 m to 90 m	0.20 mm	0.835	5.5 m	107.00 - 11.00 = 96.00 m	Higher	Scour will cease at RL 96.0 m	

$$\frac{b}{D} = 5.5\frac{d_s}{D}\left[\left(\frac{1}{11.5}\frac{d_s}{D}+1\right)^{1.7} - 1\right]$$

This relationship was found to compare well with a similar relation evolved by Laursen on basis of model test results independently. According to the latter when depth of flow was more than 5 times the pier diameter, the scour depth below river bed approximated 2.75 times the pier diameter. Work of Shen[6.16], Tarapore[6.17], Larras [6.10] and Breusers however indicates that limiting equilibrium scour depth can be less than that given by Laursen procedure. Tarapore found that when depth of flow was more than two times the diameter of a cylindrical pier, depth of scour below river bed approximated 1.4 times the pier diameter. According to Breusers relationship based on model data gives scour depth equal to 1.40 times pier diameter. For equilibrium scour with continuous sediment movement, criteria by Larras and Breusers were found by Shen to give an envelope for all known data. In view of the variation in estimated scour depth in terms of pier diameter according to various formulae, it appears safer to adopt bigger of the estimated values as obtained by Laursen curves. It is necessary to make further allowance to this scour depth to account for lowering of river bed due to constriction by guide bunds, non-uniform discharge distribution, flood scour, bed form, etc.

Out of the two methods available for scour estimation, one based on discharge in case of Lacey formula and the other based on pier size as in case of Laursen relationship, superiority of one over the other has not been convincingly proved. It is, therefore, considered advisable to work out estimated scour depth by both the procedures and adopt higher of two values.

(ii) Flood Scour

River bed surveys are usually made when river flows are low. During floods river bed may be lowered due to scour. For estimation of flood scour, riverbed is required to be gauged on the bridge section line at various flood stages and flood scour estimated by extrapolation of this data of intermediate floods to design flood stage.

(iii) Scour due to constriction

Lowering of river bed level on account of constriction of waterway by guide bunds has to be considered separately when scour depth is assessed on basis of size of the bridge pier. For estimation of constriction scour, three procedures are available which are due to Laursen, Latischenkov and Lacey [6.12]. All these being very similar, Lacey formula can be used to give

$$\frac{D_2}{D_1} = \left[\frac{W_1}{W_2}\right]^{0.67}$$

wherein D_1 and W_1 are depth of flow and width in m in unconstricted section and D_2 and W_2 are corresponding values on the bridge section within guide bunds.

(iv) Effect of bed forms

Lowering of bed level due to movement of big sized bed form is also relevant in assessment of pier scour level. Sand waves of 6 to 9 m height have been observed to form and move on bed of big rivers like Brahmaputra. When a trough passes across the pier, the depth would accordingly increase. In the absence of reliable predictors for height of bed form, field observations are needed to account for this factor.

(v) Pier scour in clayey strata

The concept of clear water scour was developed by Laursen[6.18] for application to scour at bridges on overbank spill portion where general bed load transport is absent. This concept can be considered valid also in case of scour in cohesive soils. The clear water relationship was obtained in case of abutment scour by Laursen as

$$\frac{L}{y_0} = 2.75\frac{d_s}{y_0}\left[\frac{\left(\frac{L}{r}\frac{d_s}{y_0}+1\right)^{7/6}}{(To/Tc)^{1/2}} - 1\right]$$

wherein L is effective length of the abument in m obstructing flow, y_o is the depth in m of approaching flow, d_s is depth of scour in m below bed level, r is a constant being the ratio of scour depth at abutment to that in a long contraction and was found to have value of 12, To is shear stress in kg/cm^2 on bed in the approach and T_c is critical tractive force in kg/cm^2 for the material eroded from the scour hole at the abutment. Scour at bridge piers was evaluated by substituting b = 2 L where b is width. of pier or diameter in case of cylindrical piers as was done for evaluating scour with sediment transporting flow as in Fig. 6.5. Value of To can be roughly obtained as $\gamma Y_o S$ wherein γ is specific weight of water in kg/m^3 and S is slope of approaching flow. Value of Tc for cohesive materials can be adopted from literature. Some data in this respect is given in Tables 6.5 and 6.6.

Table 6.5 : Etcheverry's Maximum Allowable Tractive Forces Given by Lane

Material	Tractive Force T_c in kg/m^2
Sandy loam	0.35 – 0.40
Average loam, Alluvial soil	0.40 – 0.50
Firm loam, clay loam	0.50 – 0.77
Stiff clay soil	1.35 – 2.12
Conglomerate, soft slate, tough hardpan, soft sedimentary rock	3.10 – 5.6

Table 6.6 : USBR Limiting Tractive Forces in kg/cm^2

Description	Compactness of bed			
	Loose	Fairly Compact	Compact	Very Compact
Sandy clays (sand content less than 50%)	0.20	0.77	1.6	3.1
Heavy clayey soils	0.15	0.69	1.5	2.75
Clays	0.12	0.61	1.38	2.60
Lean clayey soils	0.10	0.48	1.05	1.74

Laursen opined that some of the assumptions made in evolving the above relationship were rather bold. In the absence of any better approach, the Laursen relationship is, however, considered to provide at least some indication of the extent of scour in cohesive materials. Considerable judgement and caution are accordingly required to be exercised while extending the concept of clear water scour to scour in clayey bed as observed in Reference 6.19.

6.8.2 Shallow Piers

In case of some of the existing bridges, piers may be shallow and grip may be found to be inadequate. Protection round piers is then required to be given for their safety.

Minimum grip essential for safety is worked out as 1/4 the total height of existing pier from bottom to design H.F.L. At this level bed scour is required to be arrested by providing stone apron with adequate quantity to fully cover scoured face. Launching slope in sandy bedded rivers is assumed as 2 horizontal to 1 vertical. Size and thickness are assumed same as in case of apron of the guide bunds. If one man stone is found to be of insufficient weight to be stable, concrete blocks or stone crates are required to be used. Placing stone round piers at unnecessary higher level is inadvisable as it is liable to be displaced and washed down more easily. At the same time it is not a safe and feasible proposition to wait till river bed scours to just the safe level of 1/4 height to ensure minimum grip length. Dumping of stone has, therefore, to be resorted to a little earlier before the lowest safe level is reached.

Even after pitching is placed round the piers, sufficient portion of the river bed in between the adjacent piers should be left exposed so that free scour can occur. If this aspect is lost sight of, pitching of adjacent piers may practically join each other and pier protection will act as a weir. Scour downstream under such conditions has been experienced to be bigger, up to 4 times the flow depth given by Lacey formula which is dangerous.

In case of existing bridges with short spans and shallow piers, it may not be possible to avoid weir effect resulting on

account of pitching round the piers. Sufficient quantity of stone needs to be provided under such conditions to cater for full estimated scour depth on downstream side.

Similarly in case of existing bridges provided with drop walls, it may not be possible to avoid formation of hydraulic jump at certain flood stages. Provision of adequate quantity of stone on downstream side to cover scoured face by launching becomes necessary also under these conditions.

Where conditions permit, instead of dumping stone round piers, concrete blocks of adequate weight can be laid over filter covering entire area liable to scour. Protection to piers of road bridge on Godavari river near Toka in Maharashtra was of this type [6.13]. These blocks should· be laid at a level sufficiently deep allowing safe grip. Filter below blocks is essential to avoid sucking out of river bed material through gaps in blocks.

6.9 DESIGN FOR PROTECTION WORKS : INSTRUCTIONS AS CONTAINED IN IRBM PARA 818

Minor Bridges:

Most of minor bridges are on open foundation. They have to be properly protected by a well designed flooring system. This will include floor, curtain and drop wall. Length of floor and depth of drop wall will be on the basis of scour depth. This can be determined either by local observation or by using empirical value of DLacey based on design discharge. Depth of drop wall s hould be 1.25times DLacey. Floor should cover the entire width and length of abutment including wing wall. The slope of floor should match the bed slope and also the top of drop wall should match the slope. It is essential to do proper protection of the box culvert which relies on uniform ground support for its designed structural behavior. If the underside is scoured, the box culvert gets unevenly supported. For this purpose, properly designed floor system as described above should be provided. Sometimes, instead of splayed wing wall, straight return wall is provided particularly on high bank or in case of a box, another box is provided to function like a wing wall. Similar protection work is called for in such cases.

Major/Important Bridges:

As far as bridges on open foundations are concerned, it is generally on rocky/in-erodible bed and not requiring any particular protection. In other cases, flooring with drop walls as in minor bridges may have to be provided. Since well and pile foundationsare designed for the scour, hence no protection is necessary even in case of a local scour.However, bridge may need a well designed guide bund with proper protection on the approach embankments.

River training works through model studies:

In case of large alluvial river, where training/ protection works involve a heavy financial outlay, model studiesshould be resorted to, to arrive at the most economical and effective solution

6.10 DATA REQUIREMENT

Important data required to be collected for design of training works is indicated below.

6.10.1 Survey data

(i) For bridge design and training works, river plan extending at least one meander length both on upstream and downstream showing low water channel, high flood channel boundaries and extent of overbank flooding with corresponding discharges and water levels.

Cross sections at bridge site and at upstream/ down-stream boundaries of the reach with longitudinal section along the deep channel.

Layout of guide bunds, alignment of approach banks, ascertaining the necessity of marginal embankments and protection works, etc. will need such a plan and sections.

(ii) River Plan showing past available courses superimposed to examine tendency for river meanders to move downstream, to

form, cut offs or to develop nodal points. Stability of the river in plan can be examined with the help of such plan.

(iii) Specific discharge-gauge curves for a sufficiently long period in respect of both low flow as well as high flood to study stability against aggradation and degradation.

(iv) Flood cross sections on bridge alignment to assess flood scour.

6.10.2 Hydraulic Data

i) Design discharge figures for water way and foundation along with corresponding water levels.

ii) Gauge- discharge or Rating curve.

iii) Normal year and design flood hydrographs.

iv) Frequency curve for annual peak discharges.

6.10.3 Sediment data

(i) Bed material samples across the bridge alignment at surface and at various depths down to expected deepest scoured bed level so as to determine mean diameter and silt factor at various depths on basis of mechanical analysis. In case, bed material is cohesive, engineering properties like cohesion, angle of friction, Atterberg limits etc.

(ii) Suspended and bed load concentration at various flood stages along with percentage of wash load in suspended load.

(iii) Bank material samples taken during falling floods to determine nature of bank at various elevations. Samples should be analysed and results presented as in case of bed material.

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Chapter 7

PLANNING AND DESIGNING OF PERMEABLE STRUCTURES

Planning and designing of permeable structures is a new topic for the bridge engineers. Ease of construction, small cost of construction material, limited skills and experience required for the execution of the structures at site, speedy works even during emergencies, quick results, etc are the important benefits. Various topics involved for the work are discussed below in detail.

Use of permeable screens / spurs for anti-erosion work are traditional and are commonly followed in Ganga - Brahmaputra basin. In case of sediment-laden streams, it helps to induce siltation along the bank resulting in shifting of river channel away from the anti-erosion works. These methods are easy for fabrication on the nearby ground close to the site, need no especially skilled labour and can be constructed with speed. As only locally available material is used, these methods have been found very handy in the anti-erosion works during emergencies in floods. These methods also work as long-term measures in the areas where good quality stones are costly and / or are not available.

Essentially, only a dampening action on the velocity of flow is achieved by a permeable structure, distinguished from the deflecting or repelling action of an impermeable structure. The sediment transporting capacity of a flow is highly sensitive to the velocity. The theoretical considerations have shown that the weight of sediment particles is proportional to the sixth power of velocity. Therefore, the dampening of velocity results in deposition of coarser particles in the downstream direction.

The purpose, overall behaviour and layout of the permeable structures can be compared to those of submersible bund, spur and bank revetment. Permeable structures are cost effective alternative to the river training or anti-erosion works with impermeable stone spurs.

7.1 FUNCTION

Permeable structures can be used either independently or with a support of other impermeable stone structures or river training and bank protection measures. Depending upon the purpose to serve, the permeable structures are constructed in transverse or parallel to the direction of flow. Permeable structures serve one or more of the following functions.

a. Training the river along a desired course.

b. Reducing the intensity of flow at the point of river attack.

c. Creating a slack flow to induce siltation in the vicinity of the permeable structures and in the downstream reach.

d. Providing protection to the bank by dampening the velocity of flow along the bank.

7.2 CLASSIFICATION

The permeable structures can be classified as follows

a. According to function served, namely, diverting and dampening, sedimenting.

b. According to the method and material of construction, namely, bally, bamboo, tree and willow structures.

c. According to the conditions encountered, namely, submerged and non-submerged.

d. According to the type of structure provided, namely, spur type, screen type or dampeners (revetment) type.

7.3 PERMEABLE STRUCTURES

Different types of permeable structures made in the river channel to achieve the desired river training and bank protection works, viz, screens, spurs, dampeners, etc.

The permeable structures are made up of different types of smaller units called as elements. Many elements, made up of bamboos, ballies, RCC poles, etc are arranged in specific pattern and linked together to form a permeable structure.

7.3.1 ELEMENTS

Different types of elements are used for making permeable structures. The dimensions specified for the material are according to the sizes readily and commercially available in the market. Therefore, variations in the dimensions, depending up on those available in the market is made in the design.

a. Porcupines - Porcupines are made up of bamboos / ballies, have cubical shaped box at the central portion with their legs extending in all directions. The overall size is 2 m to 3 m. The central box is filled with stones for stability of individual unit during floods. Fig 7.1 shows a sketch of bamboo porcupine. Photo 7.1(A) shows a photograph of bamboo porcupine ready for laying at site.

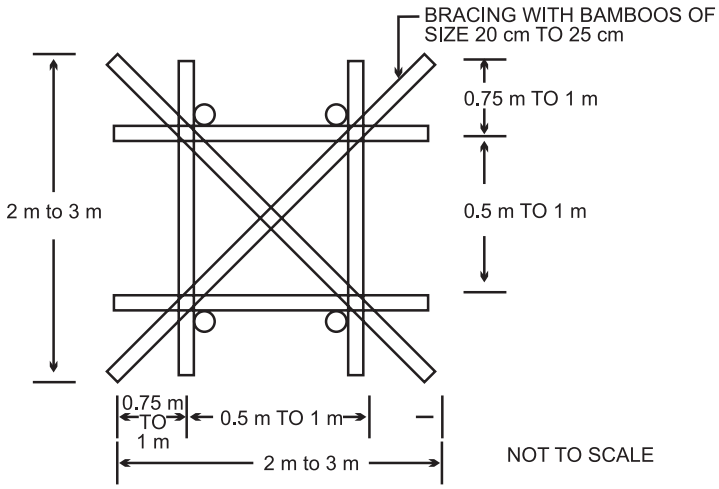

Fig. 7.1 : Elevation of Typical Porcupine

Photo 7.1 (A) : Typical Bamboo Porcupine

b. Cribs - This is a pyramid type of structure made up of bamboos / ballies with a box at the bottom for holding stones for stability during floods. Size of the box is generally square of size 2 m to 2.5 m at the bottom. Total height of the structure is 3 m to 4 m. Fig 7.2 shows a sketch of typical bamboo crib.
Photo 7.2(A) shows a photograph of bamboo crib constructed at site.

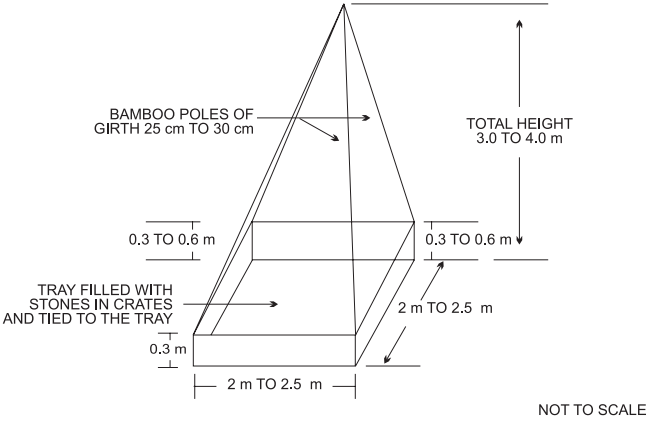

Fig. 7.2 : Sketch of typical crib

Photo 7.2 (A) Bamboo crib constructed at site

c. **Bally frames -** Permeable bally structures are made up of main skeleton of large bamboos or ballies. Cross ballies are used for stability of the structure. Photo 7.3 shows bally frame structure constructed at site, projecting from the bank into the river.

Photo 7.3 Bamboo structure projecting from the bank into the river

d. **Tree branches -** Branches of trees or trees of short height are hanged from a wire rope, duly weighted with stones and are aligned as a spur projecting into the river. The wire rope is duly anchored on the bank and in the riverbed. Fig. 7.4 shows schematic sketch of permeable structure constructed with the help of tree branches.

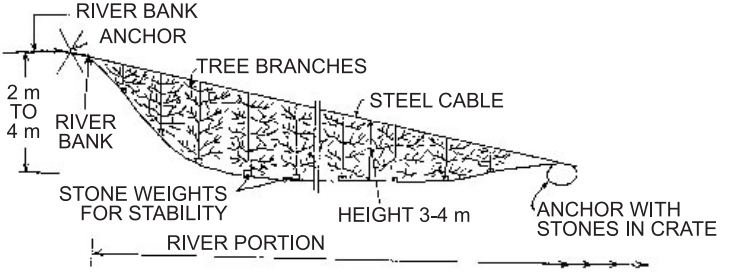

Fig. 7.4 Sketch of a typical permeable tree structure

7.3.2 Construction material

The main criteria for the selection of suitability of the material are (i) the cost and (ii) easy / local availability. Other aspects are of secondary importance. Locally available material like bamboos, ballies, brushwood, willows, bricks etc is mainly used for the construction of permeable structures. GI wire, GI wire mesh, wire ropes, nails etc are the other important but commercially available material used for the structures.

Standard commercially available bamboos of girth 20 cm to 30 cm are used for the porcupines and cribs. Smaller girth of 20 cm to 25 cm is used for bracings.

Standard commercially available ballies of girth 15 cm to 25 cm are used for the bally structures. Normally, the larger girth of 20 cm to 25 cm is used for the main members, whereas, the smaller of 15 cm to 20 cm is used for bracings

Four to 5 strands of 4 mm GI wire or wire ropes of 2 mm size containing 3-4 strands are used for interconnecting porcupines, cribs, and anchor them to the ground.

Bally driven into the ground up to a depth of 2 m are treated as anchor. Concrete anchors have an anchor rod of size 32-36 mm, well embedded in concrete cube. Wire crates anchors are of size 1.5 m X 1.5 m X 1.5 m made up of thick wires and filled with stones or bricks. A concrete block is casted with bolt and is included in the wire crate anchor. In case of emergencies, tie wires are joined directly to the wires of the crates.

7.4 DESIGN CONCEPTS

Dampening of velocities is achieved by the use of permeable structures. If the flow is sediment laden, siltation is induced in the slack flow region and the channel is shifted away from the protected reach.

Different aspects of river training and bank protection works followed for the permeable structures are similar to those followed for impermeable stone structures. For example, the criteria for location, orientation, length and number as followed

for impermeable stone spurs are followed for permeable spurs also.

The porcupines, cribs and bally structures are multipurpose elements used for all types of permeable structures. Some limitations are however imposed due to inherent weakness of the structural material and elements.

7.4.1 Selection of elements

The structural elements commonly used are the porcupines, cribs, bally frames, tree branches and willows. A suitable combination of the structure and the elements is made for the design of protection works. Following points are kept in view while selecting the elements.

The material like trees, bushes and willows are used for construction of spurs and dampeners, particularly during flood emergencies.

Permeable structures are usually designed as submersible, whereas, bally structures are generally designed as submersible or non-submersible.

In case of shallow water flows and up to a maximum depth of 3m to 4m, porcupines are used for both spurs and screens. For maximum depths of flow from 4 m to 6m, cribs are preferred. For the depths beyond these limits, bally spurs are preferred. In practice, it is observed that porcupine spurs and cribs have effectively protected depths twice those specified above.

7.4.2 Layout in plan

Permeable structures commonly used are the screens, spurs and dampeners.

Spurs made up of 3 to 4 rows of porcupines or 4 to 6 rows of cribs. Fig 7.5 shows schematic sketch of a typical permeable spur. On a straight reach, permeable spurs are spaced at 3 to 4 times its length. On a curved channel, depending up on the obliquity of flow, the spurs are spaced at 2 to 3 times the

length. Projection of the spurs into the river channel is normally 11 % to 15 %. Three spurs are provided for a specific reach to be protected. A single permeable spur is generally not found effective. Alignment of spurs is kept pointing towards upstream with reference to the flow.

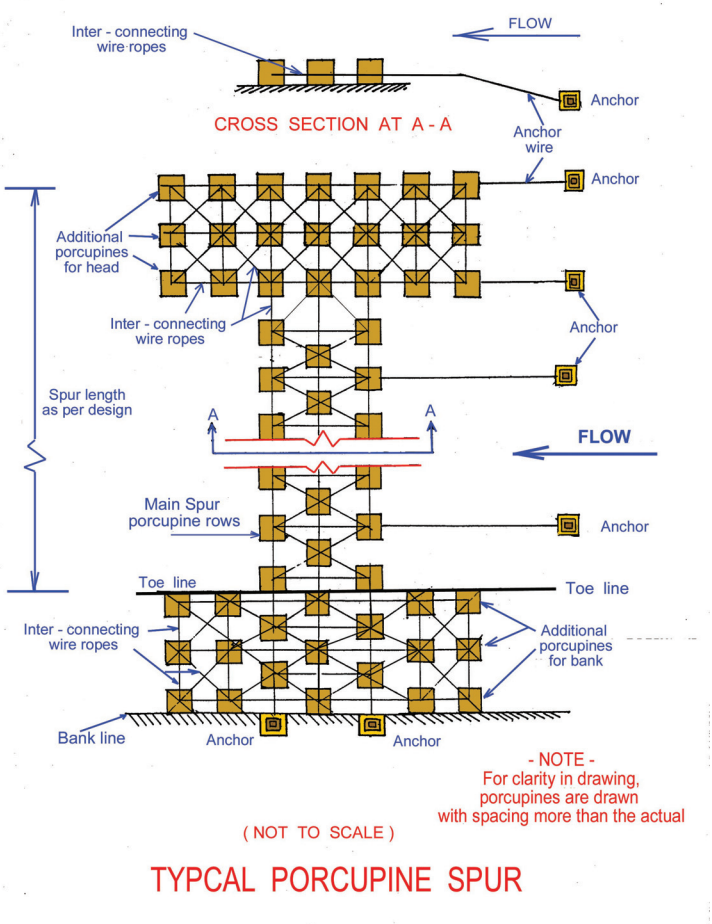

Fig. 7.5 Sketch of typical layout of Porcupine Spur

For depth of flow up to 3 m, two rows of porcupines are laid along the bank on either side of the toe as dampners. For more depth, numbers of rows are increased.

Permeable screens are used for choking the secondary channels. Four to 6 rows of porcupines or 6 to 9 rows of cribs are used in a permeable screen. One screen is normally provided at the entrance of the bypass or secondary channel. The second screen is provided at a distance of 1 to 1.5 times width of the screen and are extended on both the banks for a length one third of the channel width. Fig 7.6 shows a typical layout for inducing sedimentation in a secondary channel.

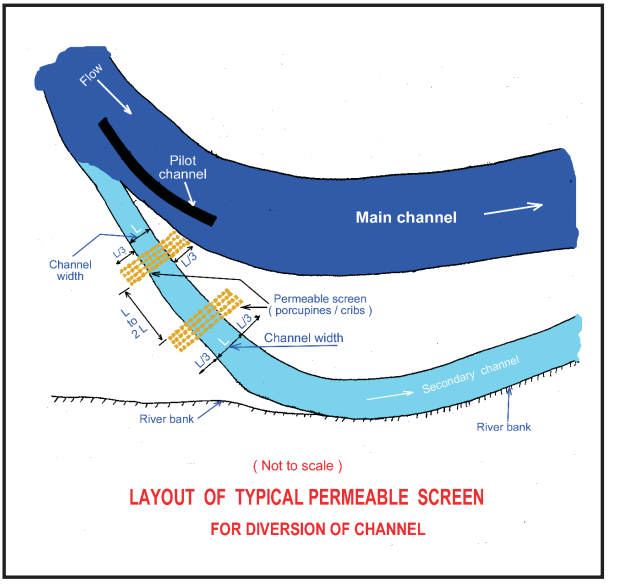

Fig 7.6 : Porcupine screens for inducing sedimentation

7.5 STABILITY OF STRUCTURES

Due to inherent weakness of the elements, the counter weights are provided (i) in the central box of the porcupines or (ii) in the bottom tray of the cribs Due care is necessary to tie the weights to the main body of the elements.

The elements are tied to each other by wire ropes. The tie ropes are duly anchored to the bank and at the nose with the help of suitable anchor or anchor blocks. Intermediate anchors are also provided at an interval of 15 m to 20 m along the length of the structure on the upstream side.

No bed protection is needed for the structures made up of porcupines and cribs. Sinking of these structures in to riverbed is a welcome feature, which adds up to the stability during floods resulting in better performance.

7.6 PROTECTION TO THE STRUCTURES

Normally, no additional protection is provided to the permeable structures. In case of estimated higher velocities along the bank, additional rows of porcupines are provided along the bank on both sides of permeable spur. Additional bed protection or protection at the nose can be provided to bally spurs. If so desired, the size of stones, size of aprons etc can be designed as per IS 8408. Temporary protection of burnt bricks in wire crates can be provided to the bally structures or to the nose of permeable spurs.

7.7 MISCELLANEOUS

In order to divert the flow and reduce pressure on the protection works, wherever feasible, pilot channels can be provided in addition to the river training works constructed with permeable spurs / screens.

In case of high velocity flows, implementation of only permeable structures is not favoured. However, use of permeable spurs in between the reach of two solid stone spurs is more effective.

Care is necessary to see that the size of the stones / bricks (the minimum dimension) is larger than the maximum size of openings provided in the crates of GI wire / nylon rope mesh.

As alternative material, instead of bamboo porcupines, RCC poles have been successfully used as anti-erosion measure

and sedimentation device. The RCC porcupines are able to withstand higher velocities. They also have longer life than the bamboo structure. Fig 7.7(A) shows a sketch of typical RCC porcupine. Photo 7.7(B) shows photograph of RCC porcupine laid at site. Photo 7.7(C) shows the porcupines laid across a large channel to induce sedimentation.

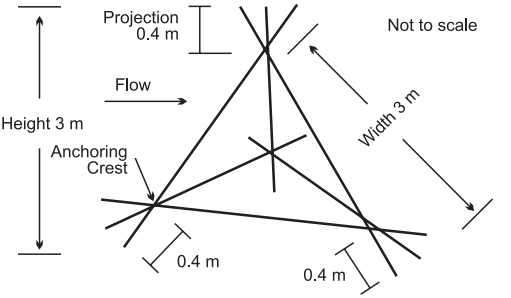

Fig 7.7 (A) : Sketch of typical RCC porcupine

Photo 7.7(B) : RCC Porcupine laid at site

Photo 7.7(C) : Porcupine screen laid across a channel

Chapter 8

FILTERS BELOW STONE PROTECTION WORKS

Percolation of water into the ground or embankment takes place, causing rise in water table and / or saturation of the body. The percolation results in seepage force in the form of pore water pressure. Thee seepage may result in to in piping through the body of an embankment in the long run. If measures are not taken to drain the water safely out of the body, failure of bank or embankment slopes during floods can take place very quickly. In the preliminary stage, signs of such failure can be seen as slips, sink holes, boils, settlement, etc.

The desired smooth and efficient drainage of water is achieved by the use of a layer of more permeable material called filter.

8.1 CONCEPT OF FILTER DESIGN

The material used for filter should satisfy the following normal criteria

a. Filter material is more pervious than the base material / core material

b. Filter material has such gradation that the particles of base / core material do not migrate through. Such migration can clog the voids and fail the very purpose of a filter.

c. Filter material helps to form natural graded material in the zone of base material / core material.

d. Normally, natural material to satisfy the above require ments is not readily available in nature. A graded filter design consists of layers of different grades.

e. To minimise segregation, each grade of the filter

material is normally very uniform in size.

8.2 FILTER BELOW THE REVETMENT

A filter placed between the revetment / protection works at the surface and the soil bed below the revetment / protection works. Main function of a filter below revetment is to allow the ground water to pass through the filter layer but arrest the movement of soil particles of the bed material.

Hydraulic conditions of the river, characteristics of bed / bank material, etc are studied before deciding the properties of filter. Some of the important variables can be listed as, location of filter i.e. whether on slope or on bed, hydraulic gradient through the filter and its direction, characteristic properties of sub-soil, water permeability of sub-soil and soil tightness, suspended sediment load in the flow and its characteristics, etc.

8.3 DESIGN OF FILTERS

There are two types of filters in use. (a) Graded filter and (b) geotextile filter

8.3.1 Graded filter

Use of graded filter has been traditionally used below the revetment and protection works. The material used in the granular / graded filters is normally durable. The graded filter material has good surface contact with both the sub-soil and the top protection works. Under gradually changing conditions, sometimes, the graded filter can show "self healing" results.

However, the graded filter requires strict quality control in design and execution. Procurement of suitable material, and maintaining the desired uniformity of filter material can be a problem. Therefore, the quality of the final product is not certain. Graded filters of more thickness show spreading effect causing damage to the protection works on the top.

Design of graded filter depends upon many factors, which are site specific. The designer also can have latitude in the design of graded filter like total filter, permeability of individual layers,

etc. Bridge engineers are suggested to use the IS code No 9429 for the design of graded filters. Due to high flexibility available in the design, each case is normally tackled separately.

Study of literature indicates simple criteria to design graded filter using a mixture of suitable grain sizes. The relations indicated in the literature areas below

$$\frac{D_{50}\text{ (filter)}}{D_{50}\text{ (sub-soil)}} < 40$$

$$5 < \frac{D_{15}\text{ (filter)}}{D_{15}\text{ (sub-soil)}} < 40$$

$$\frac{D_{15}\text{ (filter)}}{D_{85}\text{ (sub-soil)}} < 5$$

Instead of undergoing elaborate designs and construction of graded filter, mixture of 20 mm to 10 mm stone metal or brickbats in 75 mm to 100 m thickness is laid below the protection works. Sometimes kankar in the same range of sizes, screened from river sand, is used. In many cases, such short cuts have proved even damaging to the protection works. In such cases simple relations indicated above can be applied and the filter material can be made.

8.3.2 Geotextile filter

Geosynthetics is a collective term applied to thin, flexible sheets incorporated in or about soil to enhance its engineering performance. The word 'geo' implies the end use associated with improving properties and performance of engineering works founded in soil, whereas, the word 'synthetics' implies that the product is made from man-made polymers. Allied products related to geosynthetics are geomembranes, geotextiles, geogrids, geomeshes, geonets, geomats, etc, which are applied in the foundations under various conditions.

Geotextiles are textile fabrics, made up of polymers, and are permeable to water. The fine holes or pores in the geotextile allow the fluids to flow. Due to tensile and warp strength of the geotextile, geofabric filter has an advantage In case of uneven settlement of small magnitude. However, in case of significant settlements, damage to the filter is certain. Any type of damage to the filter is difficult to repair.

Comparatively the design and construction of geotextile filters is much simple. The concept of geotextile filters is comparatively new and not much known to the bridge engineers. Therefore, The geofabric filters, their designs, etc is discussed below in details.

8.4 DESIGN CONCEPT

Geofabric textiles are divided in to two types : woven and non-woven. The woven textiles are made by traditional weaving methods in which parallel sets of filaments are interlaced orthogonal to form a coherent textile structure. The size of pores in a woven fabric is pre-determined. Photo 8.1 shows internal structure of typical non-woven and woven geofabric filter.

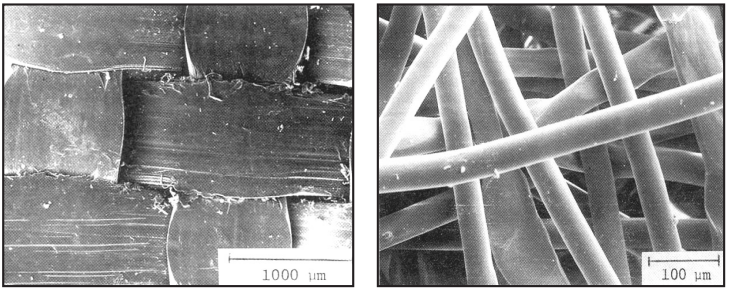

Photo 8.1 : Internal structure of typical non-woven and woven type geo-fabric

When the soil particle size is very fine, the pore size, worked out by designs, can be less than that provided by the weaving system. In such cases, non-woven geotextile is used. The non-woven textile is made up of fine and long filaments or fibers of different lengths. The material is laid in specific thickness and are "needle punched" to mix them thoroughly and form a

fully interlaced layer and pressed to form a textile of uniform thickness. A strong bond between the fibers is formed in the process.

8.4.1 Properties

A variety of material are used and processes followed in manufacturing of the geotextile results in a range of physical, hydraulic and mechanical properties. These are briefly discussed below

Physical properties normally cover the thickness (measured in mm), mass per unit area (measured in grams/sq m) and flexibility, which is measured as length of 25 mm strip to give a predetermined deflection at its free end.

Mechanical properties normally cover durability, tensile strength, puncture and burst strength, tear strength and frictional properties. Durability covers ageing due to oxidation, chemical attack by acids and alkaline materials found in soil and water, etc. Tensile strength is uniaxial tear strength of the geotextile specimen. Tensile strength is normally given in N /cm^2 or kN / m^2. Puncture and burst strength measure the resistance of the geotextile specimen normal to the plane of the specimen. Tear strength is actually tear propagation strength, measuring the resistance of the geotextile specimen to tearing after a rupture is developed in the edge of the specimen. Frictional properties of the geotextile specimen help to transfer the load from soil to the geotextile and vice versa.

Hydraulic properties of the geotextile influence its ability to function as filters or drains. The most important hydraulic properties are the size and distribution of pores and water permeability. Just like grain size distribution curves are prepared for soil, pore distribution curves are prepared for the geotextile filter. Permeability is the ability of the geotextile to transmit water across its thickness normal to the plane of the geotextile.

The designer is normally required to decide the specifications and desired properties, and select the most suitable product, which satisfy the desired standards.

8.4.2 Design criteria

The following simple criteria may be used to select the correct filter fabric :

(a) Granular material containing 50% or fewer fines (0.074 mm) by weight the following ratio must be satisfied :

$$\frac{\text{\% passing size of bed material (mm)}}{\text{opening (pore) size of fabric (mm)}} > 1$$

In order to reduce the chances of clogging, no fabric is specified with an equivalent opening size smaller than 0.149 mm and equal to or less than 85% passing size of the bed material.

(b) For bed material containing at least 50% but not more than 85% fines by weight, the equivalent opening size of filter should not be smaller than sieve No.100 (0.149 mm) and should not be larger than sieve no.70 (0.211 mm).

(c) For bed material containing 85% or more fines, filter should not be provided directly on the bed material. A cushion of fine sand of thickness 10 cm is necessary below the geofabric filter.

It may be added here the technology of producing woven fabric of finer mesh is developing fast. Therefore, the simple criteria indicated above can be superseded in a short period.

8.5 METHOD OF CONSTRUCTION

10 cm thick layer of sand having D_{50} between 0.2 to 0.6 mm is to be laid between base layer and filter. The size of filter opening should be smaller than D_{85} of sand layer. It would be preferable to use a fabric with an opening as large as allowed by the criteria mentioned above.

Over the synthetic filter, a 15 cm thick layer of coarse sand and / or gravel should be provided before placing of stones. Photo 8.2 (A), 8.2 (B) and 8.2 (C) shows the geofabric filter being laid below stone pitching at site.

Photo 8.2 (A) : Showing geo-fabric being laid below the slope pitching

Photo 8.2 (B) Geofabric filter being laid below the stone pitching. Cushion of sand is laid before spreading the geofabric filter.

Photo 8.2 (C) : Geofabric filter being laid below the stone pitching. Cushion of sand is laid on the top of filter.

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Chapter 9

HYDRAULIC MODELLING

9.1 MODELLING TECHNIQUE

Modelling is neither new nor relevant to hydraulic engineering only. It is practiced in the field of hydraulic machinery for testing pumps and turbines, in ship hydrodynamics for studying ship characteristics and in aircraft industry for testing aerofoil shapes. Modelling can be either physical or conceptual. Hydraulic and structural models are illustrations of physical modelling while mathematical and stochastic models are examples of the second category. Hydraulic modelling has become popular because of its several inherent advantages. Design conditions are rarely obtained in nature. Even if they are sometimes experienced, their duration can only be short. It is, however, possible to impose design conditions on the model any number of times and sufficiently long to permit thorough testing. Models are also useful in diagnosing causes of failures. New ideas can be easily put to test in a model but not in a proto-type. In view of these over whelming benefits, hydraulic modelIling has been accepted and practiced universally.

9.2 SIMILITUDE REQUIREMENT

When model is required to provide an aid merely for visualisation, scaler reproduction of form is sufficient which is termed Geometric Similarity. If in addition reproduction of fluid motion is desired, laws of Kinematic Similarity are required to be followed. Study of hydrodynamic fluid forces is, however, more useful when Dynamic Similarity is required to be achieved in the model, conditions for which are two fold. One is that the ratio of each one of the forces in the model to that in the prototype should be the same. The other condition is that in both the model and the prototype, sum total of all forces should be equal to inertia force.

With turbulent flow in channels, gravity force predominates. Ignoring other forces and applying the above conditions, it is then found that the Froude Number (F_r) equal to $V/(gL)^{1/2}$ should be the same in model and prototype wherein V is the mean velocity, g is the gravitational acceleration and the L the length parameter. In laminar flow phenomenon, the viscous flow predominates and the conditions obtained is that Reynolds Number R equal to VL/ v wherein v is the kinematic viscosity should be the same. Surface tension achieves importance in certain situations such as weir flow with small depths. The condition that then emerges is equality of Weber Number (W) given by $V^2 (\sigma / \delta L)$. In this σ is the surface tension and δ the density. In water hammer problems elastic compression is an important force. Couchy Number C equal to $V/(K / \delta)^{1/2}$ should then be same in model and prototype in which K is the bulk modulus of elasticity. Each of the variables in above quantities should have same basic fundamental dimensions of length, time and mass / force so that the number (Froude, Reynolds, etc.) is dimensionless.

9.3 MODEL SCALE DESIGN

Knowledge of dynamic similarity conditions enables evaluation of model scales. Almost all the problems related to river hydraulics are about gravity flow and hence required to follow the condition of equality of Froude Number. In evolving model scales for such problems it is logical to classify models of different types and consider them separately. First broad classification is according to nature of boundary- rigid boundary and mobile boundary models. Next grouping in each category can be according to geometry - whether geometrically similar or dissimilar on account of vertical exaggeration.

9.3.1 Rigid bed models

Models with rigid boundary and geometrically similar form are used in studies connected with outlet works, stilling basins, spillways etc. Conditions of Froude Number equality, equality of horizontal and vertical scales and adoption of some flow formula enable working out of the scales of the

model. The scale of boundary friction in this case works out to be such that model is required to be made smoother than the prototype.

Rigid boundary models are often constructed for determination of flood levels in rivers and flood routing studies. If geometrically similar model is constructed, depth, velocity and Reynolds Number in the model becomes too small whereas discharge required becomes too big. These difficulties can be surmounted if depths are exaggerated in the model. Such models are, therefore, termed vertically exaggerated models. Since long lengths of rivers are normally required to be reproduced in such models, availability of space usually puts constraint on horizontal scaling. Vertical exaggeration results in model roughness being required more than in the prototype which is often achieved by embedding artificial roughness elements. This puts another constraint on scaling. Exaggeration of depths in relation to horizontal dimensions is generally high in such models.

9.3.2 Mobile bed models

Mobile bed models can again be either geometrically similar or vertically exaggerated. Models of the former type permit study of problems of local scour at piers, abutments, spurs, etc. Model scale design is similar to that in case of rigid boundary model except the equality of T/T_c ratio is an additional condition to be satisfied for reproduction of sediment transport. This simple approach is based on the concept that sediment transport is governed primarily by T and T_c values.

In river models where bed changes are required to be studied, mobile bed vertically exaggerated models are used. Scale design in this type of models can be made on basis of Lacey formulae for width, depth and slope. Alternatively more rational procedures due to Einstein[9.2], Grade[9.3] and others endeavor to satisfy several conditions in addition to Froudian and flow resistance laws for reproduction of specific aspect of sediment transport phenomenon such as incipient motion, bed forms, bed load transport suspended load distribution,

local scour, channel pattern[9.4] etc. Vertical exaggeration in river models normally found suitable on basis of above conditions is about 3 to 5.

9.4 MODEL LIMITATIONS

Due to vertical exaggeration, model slope automatically becomes steeper than in the prototype, this slope exaggeration being the same as the vertical exaggeration. When river has a very flat slope even such exaggerated slope in the model may be found to be insufficient to cause satisfactory bed load movement. The model is then made steeper than the slope obtained due to vertical exaggeration and it is called tilted. Complications arising out of this expendient like non-reproduction of backwater heights and lengths are required to be taken care of separately. Complications arise even due to vertical exaggeration alone. Scour holes are not correctly formed since side slope of scour hole in the model cannot become steeper as angle of repose of bed material can't be very different. Weir coefficients become higher in the model since shape of weir crest is vertically exaggerated. Pier contraction effect becomes excessive in the model because the height of pier is exaggerated. Suitable corrective measures are introduced to overcome such difficulties and where this is not possible, allowance for divergence in the model from the proto type is made in interpreting model results.

9.5 DIMENSIONAL ANALYSIS

Modelling technique is sometimes resorted to for finding out the general law of physical phenomenon such as flow over a weir or scour at piers. Dimensional analysis provides a simple and rational approach in such studies with the use of the p theorem. In general a system may involve n variables which can be expressed in terms of m dimensional units. Buckingham's p Theorem states that the general physical law can then be expressed as a function of n-m dimensionless p terms, and each of such p terms will have m+1 variables of which only one may be changed from term to term. Once the relevant p terms are determined and the structure of functional relationship governing the physical

phenomenon is conceived, model experiments can be programmed and conducted to determine the coefficients and indices entering into this relationship.

Besides scaling of models, several other features such as model construction, model operation, instrumentation and interpretation of model results are also important and all of them need careful attention.

Important literature on the subject is cited under references.

9.6 REQUIREMENT OF FIELD DATA

Important design features of major bridge projects should be tested and finalised by hydraulic model studies. Bridge waterway, shape, length and layout of guide bunds, along the bridge and approach banks, distribution of discharge in various spans of the bridge, additional training works required if any are such aspects which are normally studied in a hydraulic model. Field data is required to be collected for this purpose which is listed below[9.10].

9.6.1 General

Preliminary report of the bridge project.

9.6.2 Survey data

(i) Plan of the river extending over 2 meanders upstream and one meander downstream of bridge site. In case of a straight reach length of river reach may be at least 4 times the khadir width on upstream and 2 times on downstream. Tentative bridge location, guide bund layout, approach banks, marginal embankments, existing training works if any etc. should be shown on this plan. Edges of low water channels should be marked.

(ii) Cross sections on bridge line, at upstream, down stream· boundaries and in the intermediate reaches. Spacing of sections should be such that in the model, distance in between them would not be more than about 3 m.

Positions of sections and their zero chainages should be shown on the plan. The levels should be close enough to enable demarcating deep and shallow portions of the channels but can be wide apart where variations in levels are gradual.

Cross section at bridge alignment should be observed at low water, medium flood and at near high flood stage.

(iii) Longitudinal section of the major channel along deep portion, termed Thalweg, Position of L section line should be shown on the plan giving periodical changes.

9.6.3 Hydrographic data

(i) Design discharge for bridge waterway and foundation with corresponding high flood levels.

Frequency curve for annual peak discharges.

(ii) Gauge-discharge or rating curve at bridge site showing rising and falling limbs separately.

(iii) Hydrographs pertaining to normal year and also the one containing peak discharge for foundation design.

(iv) Rating curves should also be observed near the cross section at model boundaries. Alternatively low water and high water slopes should be reported.

9.6.4 Sediment data

(i) Details about weighted mean diameter and silt factors or river bed material on bridge section at the positions of abutments and piers. This data should be observed at river bed and at various depths below, down to maximum estimated scour depth.

(ii) Engineering properties like cohesion, angle of friction, Atterberg limits etc., of bank material under saturation conditions during falling floods. This data

should be observed near bed level, at mid depth and near H.F.L. on bridge line.

(iii) Suspended sediment size and concentration at medium and high flood stages.

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Chapter 10

MEASUREMENT OF RIVER WATER LEVELS AND DISCHARGES

In designing bridges and other engineering structures on rivers, maximum discharge and highest flood level form essential data for design purposes. If the structure is meant for harnessing river water supplies, discharge and water level observations may be required to be made more often and in certain cases even daily. Various methods used in observation of water levels and measurement of discharge are described below, based primarily on the Indian Standards cited under reference. However, for making "short term" and "long term" measurement of discharge in accordance with Khosla Committee's recommendations, proposed Manual by R.D.S.O. for "Estimation of Design Discharge" may be referred. For methods of gauging, measuring rainfall and taking discharges on bridges nominated by R.D.S.O., instructions issued by R. D. S. O. may be followed.

10.1 WATER LEVEL OBSERVATIONS

10.1.1 Site requirements

Site for installing gauges should be properly selected. The reach should be reasonably straight and of uniform cross· section and slope. The length of such a reach should not be less than 400 m and need not be more than 1600 m. When the length is restricted upstream of the measuring section, it should be double of that on the downstream side. When near a confluence, the site, if on a distributory, should be beyond backwater effect and if on the mainstream, should be beyond the disturbance due to tributary or structures like weirs. The site should be easily accessible even during floods.

10.1.2 Different types of gauges

The gauges can be either of the two categories, non-recording or recording types. The simplest non-recording gauge is the staff gauge. It can be erected vertical in one piece or often in pieces of one or two metres each suitably spaced so that

there is some overlap of the successive gauges. In order to obtain more accuracy, gauges can also be fixed sloping if site conditions permit.

Chain, wire and tape gauges are fixed on structures like bridges and the weight is lowered to touch the water surface. Movable stilling well and electrical arrangement for detecting contact of weight with water are useful gadgets in this type of a gauge.

Pneumatic gauges permit reading or recording water level at some distance from the river, say up to about 305 m. In a diaphragm type gauge, water presses against metallic diaphragm fixed to a metal air bell. Air inside the bell changes its pressure with the variation in depth of water column above the diaphragm the air bell is connected by tubing to a dial indicator which can be located in a cabin at a safe distance from the bank. In a bubble type pneumatic gauge, the bell is provided with a slot. The air pressure is read by manometer and converted to depth of water and thus the water level is obtained. Electrical long distance gauges are also available of different forms.

In recording type gauges, automatic float operated gauges are more common. The well for the float provides room for installation of the recorder, working of float and a counter weight. Outside disturbances are damped by restricting the cross sectional area of the inlet pipe to 1/1000 of the inside horizontal area of the well. To minimize errors arising on account of lag of stylus and due to submergence of float, care is required to be taken to reduce friction of using large size float and light weight float line. More details of the float well are available in reference number 10.26.

Pneumatic gauge installation does not need a gauge well. Bubble gauge of this type gives precise observations, covers great range of variation in water level and operates over long periods up to three or more months continuously without attendance. Electrical long distance gauge recorders and radio telemetering system are also available.

10.1.3 Crest gauges

In case of bridges, it is sometimes necessary to observe high flood levels rather than daily gauges to ensure that minimum clearance is not encroached upon. Very high peaks may occur any time during day and night. Unless recorder type gauges are installed it is difficult to obtain peak gauge data. Devices called crest gauges are useful to record peak flood levels under such conditions. These can be of several types as indicated in Fig. 10.1 and 10.1 A

A simple crest gauge consists of bottles fixed to a staff gauge. Another version is an empty pipe with inside staff gauge on which peak water level is marked by granulated cork powder deposited inside the pipe. One more type of crest gauge has a float moving inside a transparent tube, the float being prevented from moving down with receding water level by means of a rubber skirt attached to it. Another simple device is to paint stripes on the staff gauge by yellow dextrine powder which leaves a high water mark when it gets dissolved in water. Details of such peak water levels over a period of years collected at the -bridge site will help in flood frequency studies.

10.2 DISCHARGE MEASUREMENTS

Methods of discharge measurement can be broadly divided into four categories: velocity-area method, slope-area method, stage discharge relationships and other methods.

10.2.1 Velocity - Area Method

Out of the above four categories, velocity area method[10.1] is most widely used and provides more accurate results. Requirements of a good site in this method are more or less similar to those in case of gauge observations. The discharge site after selection is marked by means of masonry or concrete pillars on both the banks.

Procedure adopted in this method is of observing depths and velocities on a number of verticals across the section. Measurement of depth is made at intervals close enough to define the cross sectional profile accurately. Velocity observations are

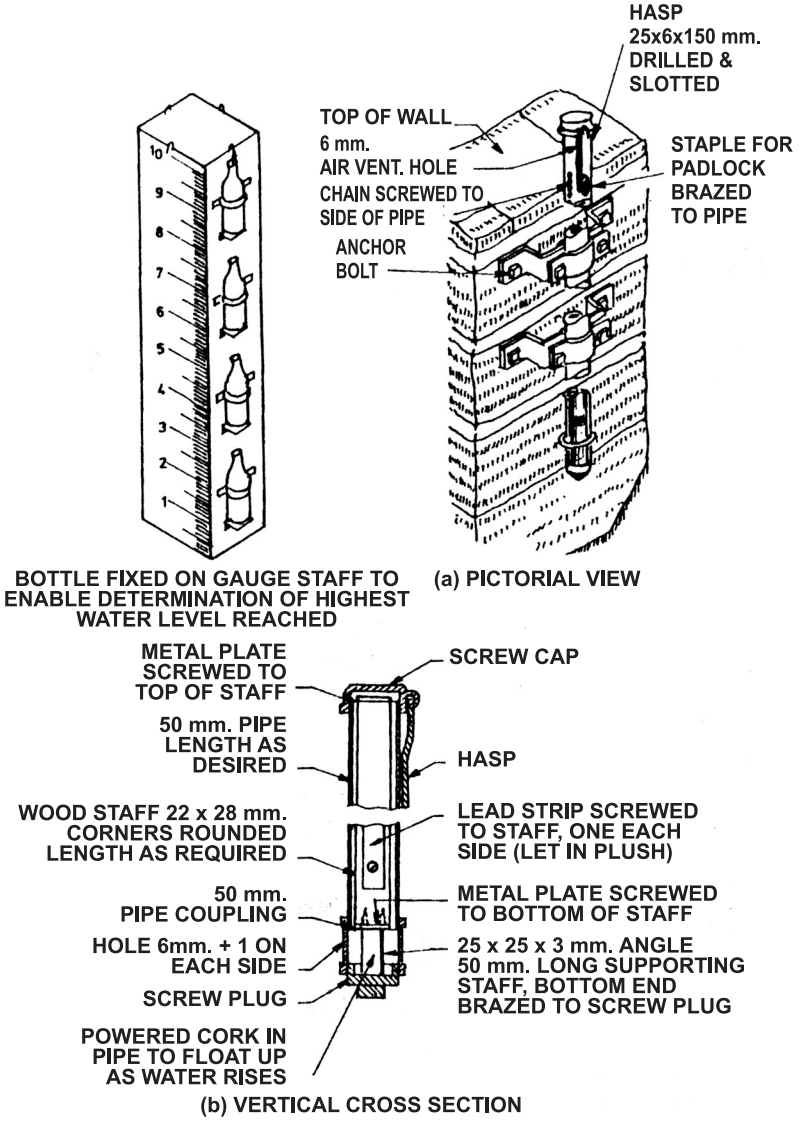


Fig. 10.1 : Staff - in - pipe type crest - stage gauge

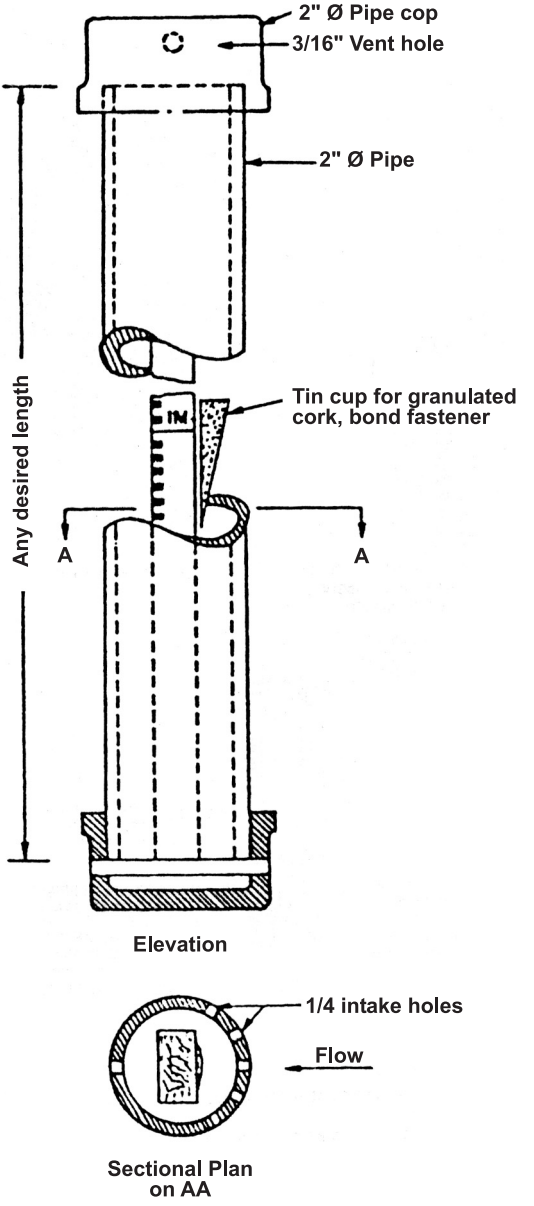


Fig. 10.1 A : Crest gauge

normally made at the same time and on the same verticals used for depth measurements. It is considered safe to normally use 25 equidistant verticals for depth and velocity observations. In the case of a river, the following values of relative errors at 68% confidence limit are given for progressively reduced number of verticals.

Number of verticals	Relative error
25	1.7%
20	2.2%
15	3.0%
less than 10	more than 4.5%

For artificial channels, smaller number of depth and velocity verticals than in case of rivers can be adopted to restrict the relative error to 2 percent.

Where the depth and velocity measurements are out of necessity made at different times, the velocity observations are made ordinarily on 15 to 20 verticals. The width of segments is not allowed to be less than 0.25 m and not greater than that in which the mean velocities in two adjacent verticals differ by more than 20 percent with respect to the lower value of the two.

Various methods are available for marking segments, of which pivot point layout is more common which is shown in Fig. 10.2.

When velocities are lower and depths smaller up to about 0.9 m, wading observations can be made using wading rods. When depths vary between 0.9 and 4.6 m, sounding rods can be used while for still bigger depths hand line consisting of a cable is convenient to use. When weights in excess of 13.6 kg are required to be used in deep water, a cable line is raised or lowered by means of a crane. For depth up to 9.0 m and velocities up to 3.0 m/s, rack and pinion equipment can also be used in boat measurements.

Even though sounding weights [10.14] are of stream lined shape, the drag on them and on the sounding cable causes

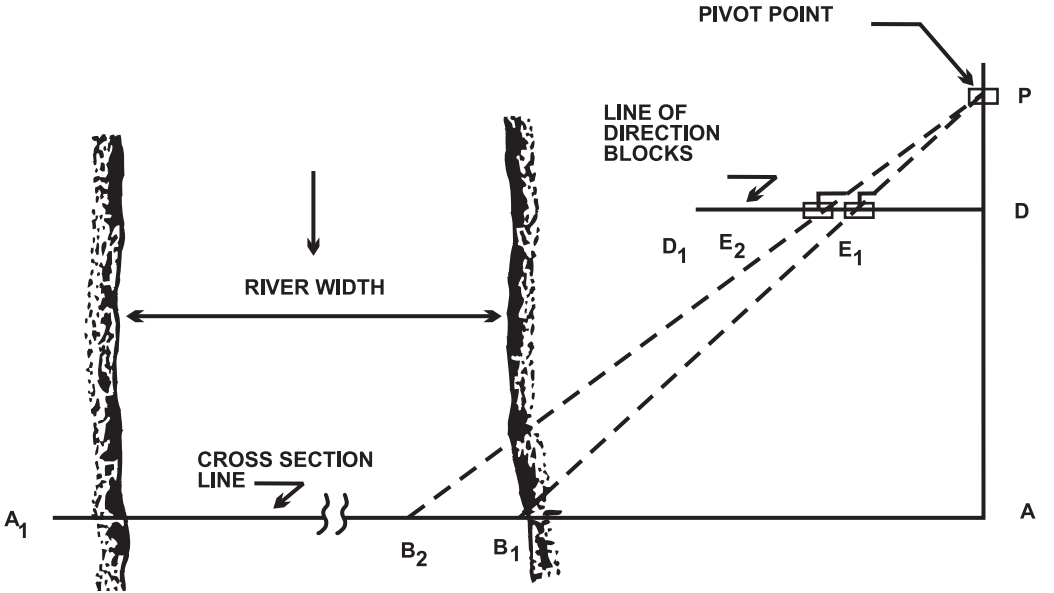


Fig. 10.2 : Pivot point layout

deflection on the suspension line from true vertical in bigger depths and higher velocities. The conventional air and wet line corrections are then applied.

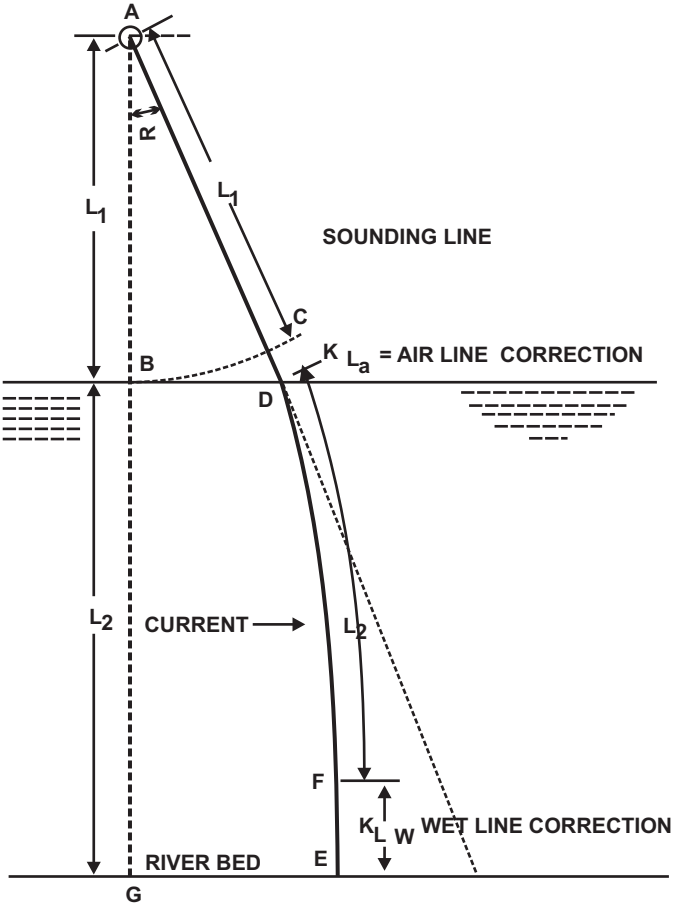
Introduction of these errors due to drag and deflection of cable will become manifest from Fig. 10.3.

The percentage correction which should be deducted from the measured length of the sounding line above water surface, called the air line correction, is given in Table 10.1.

The wet line correction, also expressed as a percentage to be deducted from the measured length of the sounding line below water surface is given in Table 10.1 for angles up to 30°.

Table 10.1 : Air Line Correction and Wet Line Correction

Vertical Angle	Air Line Correction	Wet Line Correction
4°	0.24	0.06
6°	0.55	0.16
8°	0.98	0.32
10°	1.54	0.50
12°	2.23	0.72
14°	3.06	0.98
16°	4.03	1.28
18°	5.15	1.64
20°	6.42	2.04
22°	7.85	2.48
24°	9.46	2.96
26°	11.26	3.50
28°	13.26	4.08
30°	15.45	4.72



$K_{La} = (\sec\alpha - 1)\, L_1$

Fig. 10.3 : Air line and wet line corrections

Significant errors may result if the vertical angle is more than 30°.

Echo sounders do not need such corrections and facilitate making depth measurements more easily and quickly.

The echo sounder works on the principle of sending out a pressure wave, receiving back the echo i.e. reflected wave and measuring and translating the interval of time in between in terms of depth. Velocity of pressure pulse is affected due to change in density. The transducers are, therefore, mounted at a location and elevation where intensity of stream of air bubbles generated due to turbulence, wind velocity, launch speed or any other cause is minimum. The operating frequency of the transducer is chosen to obviate effect of water noises, minimise loss of energy, minimise error on account of wide sound cone and to provide measurable time lag between incident and echoed waves. When echo sounder is used for measuring depth of scour at pier or along guide bund, wide cone introduces error. To minimise this error, sonic frequencies are avoided and supersonic range giving narrow sound cone is preferred. The echo sounder can be either indicator or recording type. Accuracy obtainable is about ± 2 cm at 15m depth for the former and ±1 cm in case of the later. Accuracy· is reduced with lower depths and is affected by change of temperature. The echo sounder is, therefore, ideal equipment for measuring big depths especially when velocities are high which may introduce significant errors in sounding weight measurements due to deflection of sounding line.

Velocities are measured by current meters either of the vertical axis cup type or the horizontal axis screw or propeller type. Of these, the cup type current meter is in common use in India and the design is very similar to Gurley, Price or Watts Current meter. IS 3910-1966 gives specifications for cup-type current meter[10.11] and IS 3918-1966[10.9] gives the code of practice for use of this type of current meter. Normally the current meter is exposed at the point of measurement for a minimum of 40 seconds and the velocity is deduced from an average of 3 readings. Rating of the current meter is made in a toeing tank and repeated after 300 working hours or after a total period of 6 months of use whichever is earlier. Before the meter is used, spin test is made and the minimum spin of 90 seconds without

and 75 seconds with spring contact is ensured. Under ideal conditions the cup type meter is expected to give accuracy up to 98 percent.

Several methods are in vogue for determining the mean velocity over a vertical such as one point, two point and multipoint methods.

In one point method, velocity is observed at 0.6 depth while in two point method, two observations are made; one at 0.2 depth, second at 0.8 depth and their average taken as mean velocity. Both these methods are in common use though one point method is mostly used in India. Subsurface method entails making velocity observation just below the surface velocity by reduction coefficient which varies from 0.79 to 0.91 and depends on depth, slope and relative roughness of bed material. This reduction coefficient is determined for a particular site by actually observing surface and mean velocities during low and medium flood stages by field observations and drawing a curve of stage against reduction coefficient and extrapolating it for desired high flood stage.

For computing discharge, mid section method is preferred as it gives more accurate results. In this method, the discharge intensity on the observation vertical is deemed to be operative over half the adjacent segments. With this assumption, discharge is computed in different parts of the cross section and added up to obtain the total discharge. Correction is applied in working out the discharge if the water level varies during the period of discharge measurement.

Ordinarily boat observations are made for measuring river discharge. Cableway can also be erected for stream-gauging purposes, especially in deep river gorges where current is fast and depths too big. Bridge observations are preferred when the river has considerable spills and facility for observations using a survey launch not available or the river is otherwise not accessible during floods. The velocity in this case is measured from downstream parapet of the bridge and more number of verticals are adopted in view of variation in discharge distribution across the section on account of pier obstruction.

While measuring discharge from the bridge, according to Research, Designs and Standards Organisation, the number of verticals should be as indicated below:

Length of Span	Number of verticals in each span
Up to 6 m	1
6 to 12 m	2
more than 12 m	3

The number of verticals are suitably modified depending on time required for completing one set of observations and duration of flood assessed by enquiry or past experience.

The effect of obstruction due to piers can be allowed for by locating the verticals adjacent to the piers judiciously in the following way, depending on whether the river bed is nonscourable or scourable.

Owing to obstruction, the velocity field gets affected in the vicinity of the piers. The flow on either side of the pier becomes accelerative. When bed is inerodible scour does not however, form round the pier. At the time of making velocity observations, it is, therefore, necessary merely to avoid this region of accelerative flow and locate the positions of verticals adjacent to the pier sufficiently away. The discharge intensity 'q_1' on the vertical 'L_1' located at this distance may be assumed to be operative over the region 'W_1' between the verticals and the centerline of the adjacent pier. The discharge in the width between vertical 'L_1' and the face of the pier will, therefore, be equal to 'q_1 W_1'.

In case of rivers permitting scour hole to develop round the piers, the flow field covering scour hole gets affected. The discharge into and out of the scour hole is required to be adjudged and allowed for. This can approximately be done by assuming the velocity and discharge intensity 'q_2' on the vertical 'L_2' beyond the scour hole on the gauging station to be operative in the width 'W_2' between the vertical 'L_2', and face of the pier. The discharge in the width between vertical L_2 and the face of the pier will then be obtained as 'q_2W_2'. The extent of scour hole in plan can be

estimated on basis of depth of scour assuming angle of repose of sand as 2 horizontal to 1 vertical.

Standard forms are used for record of gauges, cross section, computation of discharge etc. These are published in ISI 1194-60[10.3]. Alternatively the forms devised by the PCC set up pursuant to recommendations of Khosla Committee may be used.

10.2.2 Slope-Area Method

In the event of infeasibility of Velocity Area method on account of either rapid rise or fall of floods or lack of equipment or any other reason, the Slope-Area method is adopted for rough estimation of discharge.

The requirements of site are mostly similar to those in case of Velocity-Area method. In addition, the length of the reach in Slope-Area method is required to be reasonably stable, free from obstruction, straight and of uniform section for a length not less than 5 times the width of the channel and in any case not less than 300 m. The surface drop in this reach is also required to be not less than 150 mm. The gauges are read to 2 mm and the slope is worked out from the average of gauge observations at either end of the reach.

The cross sectional area is measured adopting the procedure as in case of Velocity-Area method. The velocity formula commonly used is that of Manning (Chapter 4.1), the slope entering the formula being the energy slope which accounts also for the kinetic head difference. The rugosity coefficient value is adopted following Tables 10.2 and 10.3 for coarse bed material and for bed material other than coarse respectively recommended by the Indian Standards Institution.

Table 10.2 : Values of Rugosity Coefficient n for open channels with Relatively Coarse Bed Material not characterized by Bed Form

Sr. No.	Type of Bed Material	Size of Material Equivalent	Rugosity Coefficient n Diameter
1	Gravel	4 mm to 8 mm 8 mm to 20 mm 20 mm to 60 mm	0.019 to 0.020 0.020 to 0.022 0.022 to 0.027
2	Cobbles and Shingle	60 mm to 110 mm 110 mm to 250 mm	0.027 to 0.030 0.030 to 0.035

Table 10.3 : Values of Rugosity Coefficient n for open channels with other than Coarse Bed Material

Types of Channel and Description		Rugosity Coefficient 'n'
Excavated or Dredged Channel		
(a) (i) (ii) (iii)	Earth, straight and uniform: Clean recently completed Clean after weathering With short grass, for weeds	 0.016 to 0.020 0.018 to 0.025 0.022 to 0.033
(b) (i) (ii)	Rock cuts : Smooth and uniform Jagged and irregular Natural streams	 0.025 to 0.040 0.035 to 0.050
Minor streams (Top width at flood stage less than 30 m),		
(i)	Streams on plains-clean, straight, full stage no rifts or deep pool	0.025 to 0.033
Flood on plains :		
(1)	**Pasture no brush** (i) Short grass (ii) High grass	 0.025 to 0.035 0.030

(2)	**Cultivated area** (i) No crop (ii) Mature raw crops (iii) Mature field crops	 0.020 to 0.040 0.025 to 0.045 0.030 to 0.050
(3)	**Brush** (i) Scattered brush, heavy weeds (ii) Light brush and trees (without foliage) (iii) Light brush and trees (with foliage) (iv) Medium to dense brush (without foliage) (b) Medium to dense brush (with foliage)	 0.035 to 0.070 0.035 to 0.060 0.040 to 0.080 0.045 to 0.110 0.070 to 0.160
(4)	**Trees** (i) Cleared land with tree stumps, no sprouts (ii) Same as above but with heavy growth of sprouts (iii) Heavy stand of timber, a few down trees, little under growth flood stage below branches (iv) Same as above but with flood stage reaching branches (v) Dense willows, straight	 0.030 to 0.050 0.050 to 0.080 0.080 to 0.120 0.100 to 0.160 0.100 to 0.200

IS 2912-1961 [10.4] give more detailed recommendation for adoption of the Slope-Area method.

10.2.3 Stage Discharge Relationship

Regular recording of discharges over a period of time is essential for estimation of water resources of river basins and subsequent planning and utilisation. Daily discharge observations over a long period are sometimes not feasible being too expensive. The estimation of discharge is then achieved by using proper stage discharge relation.

Besides the site requirements mentioned in connection with Velocity Area method, the following additional requirements are required to be satisfied in adopting the method of Stage Discharge relationship.

The site should provide a natural physical control for obtaining a stable stage-discharge curve. In the absence of such a control the site should provide at least partial control over the major range of stage discharge curve. If natural control is not available and the stream is sufficiently small, an artificial control in the form of a low sill or notch can be provided.

As far as possible the site should be free from variable back water effect. If such a site is not available, necessary correction is applied using twin guage method mentioned later. The site is so selected as to give a significant change in stage with change in discharge. This requirement ensures sensitivity of the stagedischarge curve which is also termed as rating curve.

The rating curve is constructed on basis of observed gauge and discharge data by fitting a mathematical curve. A sufficient number of discharge measurements using Velocity-Area method are made over the range of variation of gauges. The number of observations are so made as to evenly distribute the plotting points throughout the whole range of the rating curve. The observations made at rising and falling stages are indicated separately. In erodible channels, the discharge observations are made at frequent regular intervals and the rating curve is checked and adjusted from time to time.

The rating-curves are tested for absence from bias, for goodness of fit and for shift in control. In a bias free curve an equal number of observations are expected to be above and below the curve. As a rule this is not allowed to be greater than the percentage standard deviation of the error of the discharge observations.

Usually, the gauge discharge relationship is expressed by the equation $Q = C (G-G_o)^n$ wherein Q is the discharge, C is a constant, G is the gauge height, G_o is the gauge height for zero discharge and n is another constant.

The value of G_o is determined by using the relation

$$G_O = \frac{G_1 G_3 - G_2^2}{G_1 + G_3 - 2G_2}$$

Wherein G_1, G_2 and G_3 are the three gauge heights corresponding to three discharges Q_1, Q_2 and Q_3 which are selected in geometric progression. Alternatively the standard graphical method can be used.

If control does not change, the extrapolation of the stage discharge curve over a limited range is considered permissible. Separate extrapolations of stage-area and stage-velocity curves are also made to obtain extrapolated value of the discharge. After the bankful stages the discharge of the spill portion is worked out by estimating the velocity separately for the spills. Another method used for extrapolation when second discharge site exists on the same stream is to assess the relation between the rating curves of the two stations.

Establishment of a single gauge station is not sufficient where the flow at the station is affected by conditions existing either upstream or downstream.

In such cases, twin gauge method is adopted by establishing two gauge stations, distance between them being sufficient to ensure adequate difference in water levels, not less than 0.1 m. The normal fall and constant fall methods are then used for obtaining the discharge in case of variable slope due to backwater effect. In these methods, additional third variable of fall between the twin gauges is used. In normal fall method, this fall is the one which separates the region of noticeable effect of back water in the plot of gauge-discharge data from that having no effect of backwater. In constant fall method, any suitable fixed value of fall is adopted as a third variable.

Correction is required to be applied when flow is unsteady. The true discharge of an unsteady flow is worked out from the normal steady discharge Q_o obtained from stage-discharge curve using the following formula

$$Q = Q_O\left(1 + \frac{1}{S_O V_W} \cdot \frac{dh}{dt}\right)^{\frac{1}{2}}$$

wherein S_o Is the normal slope corresponding to steady discharge, V_w the wave velocity and dh/dt the rate of change of stage with respect to time.

Forms for use of record of gauges, for computation of discharge using Velocity-Area method, daily statement of "discharge observations, daily stage discharge data, etc. are standardised.

Exhaustive instructions for adopting the method of discharge estimation by establishing stage-discharge relationship are contained in IS 2914-1964[10.6].

10.2.4 Other methods

Moving boat method has been more recently evolved and tested in the U.S.A. and holds promise in measurement of discharge of big rivers and tidal waterways. If a current meter is towed by means of a boat in the same direction as the current, the velocity measured by the current meter will be the sum of boat velocity and-the current velocity. If the current meter is towed in the direction opposite the current, velocity measured by the current meter will be the difference between the boat and the current velocities. By moving boat parallel to flow at various distance from the bank, velocity of flow across the section can thus be computed. This is the underlying principle of moving boat method. Since the whole operation takes relatively short time, the method is convenient for adoption in case of big rivers and tidal waterways. Further details of the methods are available in literature.[10.26]

Dilution methods are more suited especially in hilly streams and torrents where current meter measurements are not feasible on account of excessive velocity and turbulence, small depth of flow, moving gravel and coarse materiel etc. In chemical dilution method, the chemical may be injected suddenly only once or it may be injected continuously. The former is called

sudden injection method and the latter constant injection method. The apparatus and chemical to be used, specifications of techniques of injection, sampling and also the methods of analysis are covered in reference number 10.25.

Radio tracer technique is similar to chemical dilution methods. A known quantity of suitable radio tracer is dumped and after it is thoroughly mixed some distance, downstream, the number of click caused by the tracer in its passage are monitored on the Geiger counter at the downstream station on the river bank. If the total number of counts is N, it varies inversely as the rate of flow Q since flow with lower velocity will allow more time for counts to accumulate on the register. On the other hand N varies directly as A, the quantity of radio tracer. Thus N = AF/Q or Q = AF/N. In this expression F is the proportionality factor which is the characteristic of the isotope, the counter and the geometric relationship of the stream. Value of F can be determined by laboratory tests[10.26].

In acoustic or ultrasonic method, transducers are fixed on opposite banks of the river, position of one of the transducers being upstream on the river. They generate acoustic pressure pulse which travel across the section. The difference between speeds of ultrasonic waves in and against the direction of water is measured from which the· flow velocity can be computed on basis of which discharge is worked out.

In electromagnetic method, a magnetic field is generated by the electric current flowing through a coil fixed below or above the channel. The voltage induced in the flowing water by the magnetic field is related to discharge. Once this relationship is determined, observation of voltage permits direct estimation of discharge.

Existing structures like weirs[10.2], [10.23], sluices[10·8], drop structures[10.21], [10.22] can be conveniently used for measurement of discharge. Formulae for flow over or through such structures are well known. Considerable work has been done to determine variation in the coefficients entering the formulae due to differences in the geometry and hydraulic conditions obtained at individual structures. Some times structures like standing wave flumes [10.18], [10.19] sharp and broad crested weirs [10.20] etc. are constructed

specially for metering the water supplies. Rigid specifications about their shapes and approach and exit conditions, locations and method of measuring water levels, formulae and coefficients to be used are then required to be followed. Reference numbers, 10.2, 10.8, 10.18, 10.19, 10.20 and 10.21 deal exhaustively with standard notches, weirs, flumes, falls, drops, orifices, nozzles, ventury etc.

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Chapter 11

MATHEMATICAL MODELLING

Rapid development is taking place in recent years in the field of Mathematical modeling in the advanced hydraulics laboratories in the world. The development is aimed at the simulation of water and sediment flows; transport and dispersion of pollutants in open channels and in wide shallow water bodies using the digital computers.

The time varying hydraulic parameters in river and canals such as velocity, water level, depth of flow, concentration of sediment and pollutants, etc. and their time dependent changes, changes in bed levels, bed material composition, changes in the concentration of pollutants, etc are simulated under known conditions and are then predicted in the changed conditions after imposing the man made changes in the system.

Flexibility, accuracy and cost effectiveness play a vital role in the development of the mathematical modeling techniques and have helped to reduce dependence on physical models. In fact, in the areas like flood routing, morphological modeling, environmental modeling, etc use of mathematical model has now become unavoidable.

Eminent mathematicians developed the basic considerations more than two centuries before. Due to limitations of the tools, numerical solutions based mainly on hand calculating devices were then possible. Practical use of mathematical models then was mainly restricted to analyse the data with the help of statistical formulation and offer a solution in explicit empirical forms.

The whole scenario has changed in the last 35-40 years due to the development of computers. The mathematical modeling techniques have now been widely accepted, which offer quick, accurate and dependable solutions of hydraulic, morphological or environmental phenomena. Now, the complexity and volume of computations involved is not an important consideration.

While adopting the mathematical modeling technique, the original system of mathematical equations is simplified keeping in view the desired objectives. The terms playing insignificant contribution to the flow phenomena are neglected and a satisfactory numerical formulation is made. The modified formulation is made suitable for solution of a particular desired flow phenomena. Such modifications are then cost effective and are suitable for the speed and capacity of the computers. However, the application would also get restricted to the desired objectives kept in view by the designer.

Idea for presenting this chapter is to impart only the preliminary ideas regarding the mathematical models. The mathematical aspects like the original mathematical formulation, simplifications made, adaptation of the equation to specific objectives, etc are not discussed here. Moreover, the discussions cover mainly the hydrodynamic phenomena, which is of more interest to the bridge engineers. Other aspects, like evolving the boundary conditions etc similar for both physical and mathematical model studies. Therefore, the same also has not been covered in the discussions.

11.1 TYPE OF MODELS

Depending up on the requirements of the studies and the predominant hydraulic phenomena involved, mathematical models can be grouped in to three types.

(a) Unsteady flow conditions
(b) Quasi-steady flow conditions
(c) Steady flow conditions

Normally, steady flow conditions are considered for backwater computations, which are meant for a specific discharge stage. For morphological computations, i.e. to compute time dependent changes in the bed levels, quasi-steady flows are used and computations are made at every discharge stage. Unsteady flows are required to compute flood wave propagations, flows in tidal reaches, etc.

Depending upon the requirements of studies, mathematical models are also grouped in three types

(i) One dimensional model
(ii) Two dimensional model
(iii) Three dimensional model

One-dimensional model generally means modeling of a river along its length. All the hydraulic parameters are averaged and are assumed to act at one point. This is good approximation for, say, flood propagation in the rivers, where variation in the flow direction is generally important. In such computations, the bed friction governs the speed of propagation and depth of flow at a particular location, which is introduced by Manning or Chezy's relations.

Main advantage of 1D models is the reproduction of a network of channels in a river system. For example, Fig 11.1 shows a typical network of river channels reproduced in 1 D mathematical model. Propagation of floods can be easily reproduced in such complicated layout of river channels.

Natural changes like land slides, earth quakes, extreme floods, deforestation or man made changes like construction of

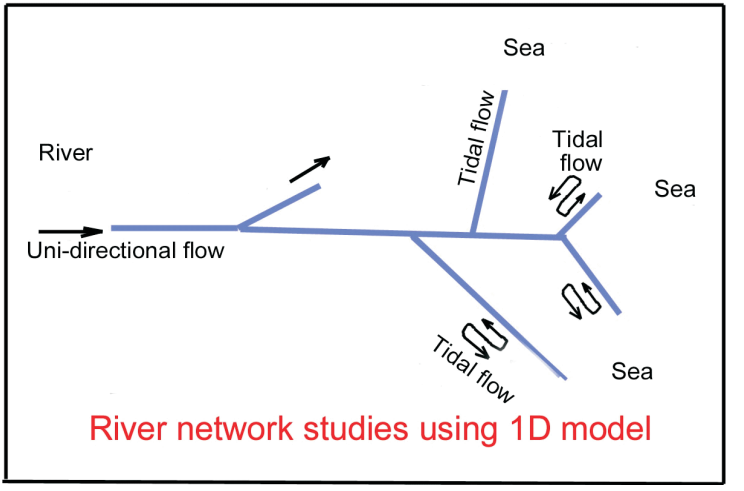

Fig 11.1 : Typical network of river channels reproduced in 1 D mathematical model

dams, barrages, bridges, embankments affect the natural regime of rivers. Changes in the river regime affect river plan form, hydraulic relations at a particular location etc. Modeling of morphological processes in rivers is covered in this type. Using quasi-steady flow conditions and sediment transport relations, time dependent changes in the bed levels can be computed in 1 dimensional model. Fig 11.2 shows bed level changes computed for an under-nourished canal for a period of about 30 years. Progressive reduction in the overall bed levels and changes in the bed profiles are apparent from the figure.

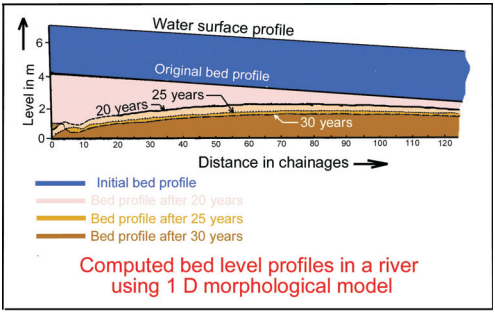

Fig 11.2 : Computed bed level profile using 1 D morphological model

Two-dimensional model can be of two types, laid in X-Y plane and in X-Z plane. Modeling in horizontal direction (X-Y plane) computes variation in X-Y direction with vertically averaged values

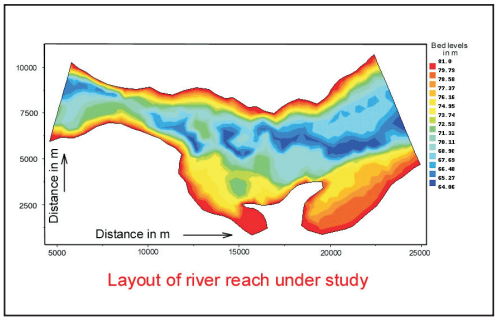

Fig 11.3 : Bed levels (topography) reproduced in a 2 D mathematical model.

of velocities at each location in X-Y plane. For bridge engineers, it is necessary to assess the effect of the location / orientation of a bridge, and the effect of constriction on both upstream and downstream of a bridge. As far as these aspects are concerned, the studies with the help of physical models could be comparable with those conducted using 2D mathematical models.

For many bridge engineering solutions, the results of 2 D mathematical models are sufficient for the design of hydraulic structures. Being in 2 D (plan) shape, good graphics help the user to get more "natural and realistic" feel of the problem / results. Few typical cases are presented here for quick understanding. Fig 11.3 shows bed levels (topography) reproduced in a 2 D mathematical model. Fig 11.4 shows velocities computed in 2 D model for a specific discharge stage for the same topography shown in Fig 11.3. The concentration of flow and higher velocities generated at the nose of a spur result in higher sediment transport and deeper scours immediately downstream of the nose of a spur. Fig 11.5 shows bed levels computed at the nose of a spur in sediment transport module of 2 D mathematical model. The scours computed in the model have been reversed for better understanding of the scours. These are some of the examples of the use of mathematical models for the studies in river engineering problems.

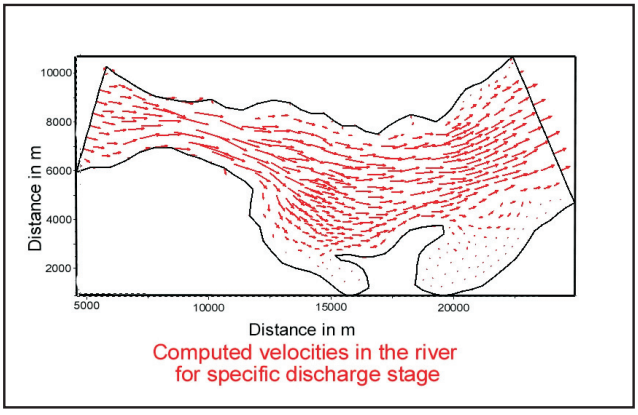

Fig 11.4 : Velocities computed in 2 D model for a specific discharge stage

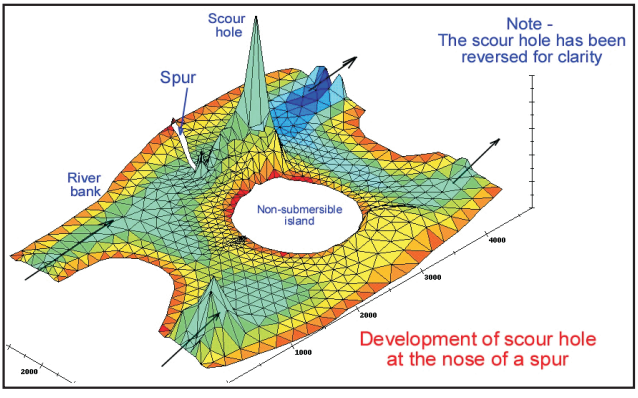

Fig 11.5 : Bed levels computed at the nose of a spur in sediment transport module of 2 D mathematical model. The scours computed in the model have been reversed for better understanding of the scours.

Modeling in vertical direction computes variation in X-Z direction, whereas, variation in Y direction is not considered. These models can be compared with the flume studies conducted as a part of physical model studies. For bridge engineers, these models may have use in specific cases like effect of bridge piers on another bridge located in the vicinity, design of complex type of piers, bridge laid at an angle to the main flow direction, etc.

The three-dimensional model considers X-Y-Z directions simultaneously. Scours around bridge piers and abutments are the typical problems 3 D in nature. For bridge engineers, the use of 3 D mathematical models could be limited in the present context.

Apart from the hydrodynamic modeling of river flows, river engineering softwares are now commercially developed to cover many other aspects like rainfall-runoff and other hydrological aspects, ground water flows, dispersion of pollutants in the flows, etc to bring other related aspects under "one umbrella". With the development of graphics and application of many analytical processes, graphical and pictorial presentation of the data, analysis and display of the computed results, animation of results at specific locations over specific period, etc has become highly interesting and easy.

11.2 SYSTEMATIC STAGES FOR THE STUDIES

The data requirement for a physical model and a mathematical model is exactly same. The development of a physical or a mathematical model undergoes the same stages as indicated below :

a. Layout of a physical system of desired reach.
b. Determination of boundary conditions at both upstream and downstream ends.
c. Determination of initial conditions.
d. Initialization of model with pre-determined initial conditions.
e. Accurate control of boundary conditions during the cali bration run / run under existing conditions.
f. Check for smooth running of the model during the whole computational domain.
g. Extraction of desired parameters during the model run.
h. Verification of results and comparison with prototype data.
i. Running the model under modified conditions as required for the prediction run and extraction of desired parameters.
j. Analysis of computed results, comparison with the results under existing conditions and evaluation of the results with modified conditions.

For transformation of the physical system into a mathematical model, the discretisation technique is followed. For one-dimensional mathematical model, the river length is broken into fragments. Storage areas, flood planes etc are also reproduced as per the requirement of the model. This part of modeling technique yet required experience and skill. Water levels and discharges are taken as "open boundaries" and suitably specified as gauge discharge curves, water level hydrographs, discharge hydrographs, etc.

11.3 ADVANTAGES AND DISADVANTAGES

A comparison between the two techniques is inevitable which would also help to select the suitable one for a particular problem posed.

a. Scale effects

Advantage of similarity of predominant forces is taken in a physical model. Thus, the remaining forces are not similar and bear scale effects. Interpretation of results needs proper interpretation and evaluation of the values of different parameters to the prototype. This has a particular reference to the sediment transport, deposition, and scour etc. Interpretation of these phenomena is more of an art than a standard technique. In a mathematical model there are no scale effects as the natural dimensions are used everywhere.

b. Type of models utilised

The physical models are used mainly for reproduction of hydraulic phenomena, which are very well described. However, reproduction of unsteady flow conditions in physical models is difficult and costly. In case of channel network system, it is even impracticable. Due to complexity of nature of sediment transport, achieving similarity of sediment transport in a physical model is not possible yet. Therefore, reproduction of year-to-year changes in a physical model is not possible with the present techniques. Mathematical models have some advantage over physical models in such cases.

c. Cost and time

Layout of both physical model and mathematical model is a matter of art. However, the time spent and cost involved in analysis of data and adoption of the data for layout of the model is much less in case of mathematical model.

Conducting model studies is both costly and time consuming in case of physical models compared to the mathematical models.

d. Establishment cost

Though the computers and their essential peripheral are costly for initial installations, the same computers are used for any mathematical model later on. Against this, costly ground spaces and other supporting devices are required for physical

model which increase the preliminary cost before taking up the construction of a physical model.

e. Model limitations

Description of 3 D flow phenomena is difficult and costly with the present techniques in a mathematical model. A physical model is a better alternative at present. Description of a morphological process of a river is possible in a mathematical model. A physical model is unable to reproduce long-term changes in the riverbed. Therefore, a combination of both mathematical and physical model is advisable for more complicated problems and problems at the delicate and sensitive areas where support of another technique is needed before finalising a decision by one technique. In other cases, any suitable alternative could be chosen for solution to a problem.

11.4 CLOSING REMARKS

There are many mathematical models available in the market covering one or more of the discussed types. Keeping in mind the advantages and limitations of the available models, they can be conveniently used for studies.

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