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A CRITICAL REVIEW OF THE DESIGN PROCEDURES OF RIVER DIVERSION WORKS

A DISSERTATION
submitted in partial fulfilment of the
requirements for the award of the degree
of
MASTER OF ENGINEERING
in
WATER RESOURCES DEVELOPMENT

By
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CERTIFICATE

Certified that the dissertation entitled "A Critical Review of the Design Procedures of River Diversion Works," which is being submitted by Shri S. R. K. Vadali in partial fulfilment of the degree of Master of Engineering in Water Resources Development of the University of Roorkee, is a record of the candidate's own work carried out by him under my supervision and guidance. As far as I know, the matter embodied in this dissertation has not been submitted for the award of any other degree or diploma.

This is further to certify that he has worked for a period of nine months from 1st October 1972 to 30th June 1973 for preparing this dissertation.

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SYNOPSIS

Irrigation is among the oldest applied arts, practised by humanity from the dawnof its history. Until late in the Nineteenth Century, irrigation was still considered an advanced form of agriculture. The concept of irrigation as a branch of Engineering Science, is, therefore, relatively recent. The necessity of abstracting water from its natural source to convey to the fields for irrigation, led to the necessity for the design and construction of irrigation works, such as weirs, barrages and dams &c.

Consequent to the failures of a number of diversion works, in the early 20th Century, the necessity for a rational approach for the design of these structures, assumed great importance. With the advancement of hydraulic science, several theories have been evolved for a safe and satisfactory function of such structures.

The direct object of any diversion structure is the control of the water flow in a river or in a canal, and, therefore, the first step in its design is the hydraulic analysis of the conditions under which it is supposed to work. The quantities dealt with in this analysis are discharges, velocities, water slopes, water levels, silt charge &c. The general arrangement of the proposed work, and its main dimensions, will be based on the result of this analysis.

In our country, however, the general approach to the design of these diversion structures differs from state to state, and river to river. The practices adopted in Uttar Pradesh, which ranks as the first in number of barrages with its vast canal system, are different from those adopted in the Punjab, who have done considerable work on the control and exclusion of silt in canals. The Central Water & Power Commission also have a different approach for the design of these structures.

In view of so much divergence in practices adopted by various organisations, an attempt has been made in this dissertation to review and summarise different procedures adopted for design of diversion structures by various authorities and the reasons which favoured their adoption, as also to indicate the various problems in this field on which further study is called for.

C O N T E N T S

Chapter 1

OBJECTIVE AND SITING

1.1. Introduction

Barrages and weirs, as distinct from storage dams, are structures constructed generally on permeable foundations to divert the run of river waters into canals for irrigation.

The evidence of earliest construction of a diversion weir in India for Irrigation can be traced back to the second century A.D., when the Grand Anicut was built of stones laid in clay across the river Cauvery in Southern India. A number of irrigation systems mostly of inundation canals, apart from tanks and small reservoirs, were built in the medieval ages. With the advent of the Nineteenth century, a number of great ventures were undertaken towards improvement and utilisation of old indigenous diversion works, when Western Yamuna Canal, Eastern Yamuna Canal and Cauvery Delta System were rejuvenated. Encouraged by the satisfactory results, classical works, such as the Upper Ganga Canal in Uttar Pradesh, Upper Bari Doab Canals in the Punjab and the Godavari Anicut across Godavari and Kistna Delta projects in Andhra Pradesh - were constructed and were later followed by the Sirhind Canal, the Lower Sonag and Para canals, the Lower Chenab Canal and the Sidhani Canal in the Punjab, Lower Ganga Canal, Agra Canal and the Betwa Canal in Uttar Pradesh, and the Jamrao and Western Nara Canals in Sind. This period marked the construction of permanent masonry weirs resting on wells, with apron in its rear for these diversion projects. One some of the works, clay apron was provided on upstream, and deep wooden sal piles and circular wells were provided at end of apron or downstream floor, for protection. The Triple canal projects, the Lower Jhelum Canal and the Upper Swal Canal all in West Punjab, Pravara River Canal and Nira Canals in Maharashtra, Anderson Weir on Damodar, were undertaken in early twentieth century. With the Sarda Canal project in Uttar Pradesh, Sutlej Valley projects in the Punjab and Sukkur Barrage in Sind, was ushered the great era of Barrages in the history of diversion works in India. The works were earlier designed on basis of creep theory when nature of failure by piping as demonstrated by Terzaghi and, later, by Khosla, were unknown. Concrete in floor, sheet piles for cutoffs, appropriate slopes of glacis, inverted filter, flexible aprons, and ancillaries in basin for destroying the energy of flowing water, are generally adopted and diversion works are now designed in accordance with latest advance in hydraulic engineering science.

The Harike Barrage on Sutlej, the Yamuna Barrage at Dakpathar, the Narora Barrage on river Ganga, Durgapur Barrage across the Damodar, Mundali across the Mahanadi, Krishna across the river Krishna, Kosi and Sone Barrages in Bihar, are a few notable examples of the major diversion works constructed in Indiawith modern techniques and engineering science.

1.2. Objective

The aims and objects of providing permanent diversion works are as follows:

i) To ensure the desired water supply into the canal for which development has been designed. This is accomplished by ponding to raise the level of water in the river. (Ponding may become necessary at some or all times to raise the water level in the river to a level with which the off-taking canal may be fed to be able to command the area to be irrigated.)

ii) To prevent or reduce the entry of silt into the canal.

1.3. Types of diversion works

These are classified into two distinct categories according to the type of control of the flow passing them:

i) Open weirs or simply weirs

ii) Barrages

The difference between the two is not rigid but qualitative.

'Open weirs' or simply 'weirs' provide the major part of the obstruction in the form of a permanent crest across the river. The length of the weir and top of its crest are determined by the discharge per metre run and permissible afflux during the maximum flood. The pond level can be maintained by a permanent masonry weir with its crest at pond level or by use of falling shutters installed on a weir with lower level crest.

'Barrage' is a weir with low crest generally at or near river bed and having gates for maintaining pond level, provides a complete control of the river channel especially in low floods.

The choice between a weir and a barrage is largely governed by economy and convenience of working. A shuttered weir is generally cheaper, but will lack the speedy and effective control possible with a barrage.

1.4. Location of diversion works

The river stages can be divided into the following:

i) Rocky stage, where the river slope is steep and bed is rocky;

ii) Boulder stage, where the river slope is still steep and the bed has boulders and shingle;

iii) Trough stage, where the river slopes are flat and the cross-section is made up of alluvial sand and silt;

iv) Delta stage, where the flow is sluggish and the river channel is divided into a number of small channels joining the sea.

Diversion structures are seldom located in the rocky stage as enough area would not be available for irrigation in hills.

Although some of the important diversion works have been constructed in delta-stage, e.g. the four anicuts on the Godavari, the Birupa and the Jobra anicuts on Mahanadi, it is seldom necessary to construct a diversion work in delta stage. Most of the major diversion works are therefore located in the boulder or trough stages.

The decision whetherthe diversion works of any canal should be located in the boulder stage or in the trough stage depends mainly on the consideration, whether the tract requiring irrigation, can be commanded by the canal off-taking from it. Although there are merits and demerits of location of diversion works in these stages, these have been successfully constructed in either stage.

The advantages and disadvantages of siting diversion works in the boulder stages can be stated as below:

Advantages:

i) River training works are few;

ii) The weir can have high intensity of discharge and cost should thus be relatively low;

iii) Power benefit can also be combined due to availablity of steep slopes in the river.

Disadvantages:

(a) A weir in boulder tract from all appearance may look fairly water-tight but still sub-soil losses from it will be high due to strong sub-soil flow through its pervious foundations. (This disadvantage is very much marked in the upper reaches where the entire water may sometimes disappear into the ground, leaving the river bed almost completely dry, maybe, to appear again lower down.)

(b) The canal will usually have a large number of costly drainage crossings in its head reach.

(c) The canal generally will have all idle uneconomical reach in its head reaches.

The advantages and disadvantages of siting the diversion works in the trough stage can be summarised as below:

Advantages:

i) The area where irrigation is required will usually be quite near and so the idle length of the canal to be constructed to reach the irrigable area is not usually long.

ii) The number of cross-drainage works is also not large.

Disadvantages:

Diversion works in trough reaches are more expensive, as apart from necessity of transporting construction materials from long distance, these require expensive training works and marginal bunds. Actually, a heavy annual expenditure on training works must be considered necessary for all big diversion works in trough reaches.

1.5. Siting of diversion works

The major considerations which govern the selection of site for locating diversion works can be summarised as below:

i) The site should be fixed in a reach where the river channel in low stage is likely to stay permanently along the bank of off-take of the canal, and the canal off-take can be located so as to be on the concave side of the curved stretch and preferably along the downstream reach of the curve near the end. (The latter criterion will permit silt exclusion devices to operate at good efficiency if more or less favourable bund conditions are created.)

ii) The banks in the reach should be stable so that the training works required are minimum.

iii) The river channel section upstream of the work should be wide enough, so that there is no serious risk of out-flanking. However, the width of the river upstream should not be abnormally large so as to lead to cross-currents and cause concentration of flow in certain portions. (In a very wide valley, holding the river to the diversion works also pre sents special problems. On the other hand, a very narrow river may mean, high intensity and heavy protection against scour.)

iv) A canal alignment free from the attacks of the river should be possible.

It is easy to combine a road bridge with the work and in such cases, the road requirement also determines to a certain extent the final fixation of a site for a diversion work within the selected reach.

1.6. Diversion works in straight reaches

A straight reach would be necessary for a diversion work, sometimes, when there are canal off-takes from both sides. A curved approach in such cases would lead to sediment problems for the canal taking off from the convex bank while the concave-bank off-take will have sediment-free water. The Sukkur Barrage across the Indus is an example of such work where the left bank canal which took off from the concave side had no sediment trouble while the right bank canal taking off from the convex bank got heavily silted up.

Chapter 2

VARIOUS COMPONENTS OF A DIVERSION WORK

2.1. The chief constituents of a diversion work are:

i) The Weir or Barrage - with undersluices and river bays separated by suitable divide walls, as also with energy dissipation devices in the basin and protection against erosion and undermining on the downstream, as well as on the upstream;

ii) The head regulator for offtaking canals;

iii) River training works;

iv) Silt exclusion devices;

v) Special features such as Fish Ladder and Navigation Lock, etc.

2.2. <u>Functions of the various components</u>:

The functions of the various components of a diversion work are as follows:

(i) <u>The Barrage or Weir proper</u>

The barrage or weir provides the obstruction across the river required to raise up its water level and divert the water into the canal. It is normally aligned at right angles to the direction of flow of the river. The flow into the canal is controlled by shutters or gates provided for the purpose on the weir or barrage.

Undersluices are an essential feature of all diversion works. These are normally located adjacent to the head regulator and are separated by divide walls from other bays.

The main functions of undersluices are as follows:

(a) To preserve defined river channel, approaching the head-regulator. This is achieved by keeping the crest of the floor in undersluices lower than the river bays. Sometimes, however, the crest is kept at the same level throughout the diversion structure.

(b) To flush the silt deposited in the pocket forming the still pond in front of the canal regulator.

The functions of a divide wall are three-fold:

Firstly, the divide wall separates the depressed undersluices from the raised crest of the weir or barrage bays. This helps in preventing parallel flow by distributing the flood discharge between the undersluices and the river bays.

Secondly, the divide wall segregates the pocket from the barrage and thus helps creation of conditions of still pond in the pocket. The siltthat settles down can be easily flushed out.

Thirdly, in the presence of divide wall, with suitable regulation, a favourable curvature of flow and ratio of V_R/V_P (where V_R = velocity on river side of divide wall, and V_P , velocity in undersluice pocket), more than unity can be obtained, to help in maximum exclusion of silt from the canal.

In most of the modern weirs, the hydraulic jump is used for dissipation of energy. To create conditions for hydraulic jump, a sloping glacis is usually provided, the slope being 1:3 to 1:5. The floor downstream of the glacis is kept at a level, so that the depth is adequate for formation of hydraulic jump at all stages of flow.

In addition, the installation of accessory devices, such as chute and baffle blocks, sills, along the floor of the basin produce a stabilising effect on the jump, which permits shortening the basin and provides a factor of safety against sweepout due to inadequate tailwater depth, if any.

The upstream and downstream floors need protection against scour. This is achieved by providing sheet piles at either ends of these floors. In addition, in the upstream next to the sheet pile lineprotection of blocks laid over loose stone followed by launching apron consisting of only loose stone,is provided. In the downstream of the pucca floor, graded filter protected by blocks, and substantial launching apron beyond the filter are provided.

(ii) Head Regulator for the off-taking canal

The functions of a head regulator are two-fold:

a) To ensure the desired water supply for which the development has been designed.

b) To prevent or reduce entry of silt to provide protection against sedimentation in canal bed.

c) To close the canal in case of emergency or lack of demand.

A head regulator is generally provided with -

i) A crest, whose elevation should be such that the minimum quantity of water required is assured.

ii) A gate control for regulation.

iii) A breast wall ifnecessary to avoid overtopping of floods into the canal.

(iii) The river training works

River training works that may be necessary at any diversion structure are:

i) Marginal bunds (or embankments)

ii) Guide banks

iii) Spurs

Marginal embankments are provided to confine the river to the cross-section consistent with the length of the diversion work, in sufficiently long reach on the upstream of the work affected by the afflux. These also prevent additional area from getting submerged due to rise in flood level caused by afflux.

Guide banks are meant to provide smooth approach to the diversion structure and to guide the river flow past it. They are sometimes aligned to create a favourable curvature to provide effective sand exclusion from the canals.

Spurs projecting into the stream from the side banks or marginal bunds, may be required to protect the marginal bunds or to deflect the current to the opposite bank, or attract it.

(iv) Silt exclusion devices

In a stream carrying sediment in suspension, the sediment load is much greater near the bed than in the middle or near the top. Also the sediment particles near the bottom are much coarser than those in the upper layers. If the bottom layers are removed without disturbing the natural sediment distribution in the stream, a large quantity of sediment carried by the stream will be removed. The withdrawal of bottom layers of flow, besides removing the sediment in suspension, also removes the coarse material which is in saltation or moving on the bed. Preventing the entry of this coarse bed sediment into the canal is the principal aim of all sediment control measures. Sediment excluders, extractors etc. are all based on this principle. A diaphragm, provided at a suitable height in the stream secures this separation.

In the barrages and weirs sediment excluders are constructed in the river pocket, which feed the channel. On rivers in the alluvial stage, the excluders deal with sand only, while on the rivers in the boulder stage, they have to exclude sand, gravel and other coarse material. Different types of excluders have been tried on different head works. Generally, the excluders cover only a few bays of the undersluices. The excluder tunnels are kept open at the front. Side openings in in addition to the front openingshave also been given in some old diversions works, but this reduces the efficiency. A double-decked excluder, first of its kind, was provided at the Nangal Barrage.

Large tunnel excluders are not very efficient as the larger the excluder, the greater will be the discharge brought into the pocket. The sediment load brought will be out of proportion to the increase in the discharge. A large discharge creates a lot of turbulence in the pocket as a result of which sediment jumps over the excluder roof and escapes exclusion.

For evolving a suitable design of a sediment excluder, the local condition, i.e. the curvature of flow, hydrograph of the river, position of the canal regulator, nature of the bed and vertical distribution of sediment in the pocket have to be studied carefully.

The river approach is one of the most important factors which influence the working of a sediment excluder. These should be carefully studied for a number of discharge conditions, both at site and by model experiments.

The number of tunnels for an excluder is determined by the discharge of the canal, conditions of approach, length of the canal regulator and the available escapage discharge. A few tunnels located at suitable positions are much more advantageous than a large number of tunnels placed without regard to their suitable positions. The excluder at Khanki Head-Works had six tunnels. It was found, later, that only three tunnels next to the regulator worked satisfactorily. At the headworks of the Western Yamuna Canal at Tajewala, the shingle excluder originally designed provided only two tunnels, which worked efficiently.

The size of the tunnel is governed by the discharge to be escaped through the excluder, the depth of water in the pocket and the velocity of flow required in the tunnels to keep them clear of any deposit.

The bed level of the tunnels is fixed on the basis of the vertical distribution of silt at the site, so that, it entraps a major portion of coarse material. The top level of the slab of the excluder is kept at the sill level of the canal regulator. The height of tunnels generally varies from a couple of metres to several metres.

Usually the excluder covers about two bays of the under-

-sluices, but it may sometimes cover the entire width. Sediment excluders covering the entire width of undersluices were constructed on Emersion Barrage (Pakistan) and on the Kosi (Hanuman-nagar).

Adequate theoretical background for designing all these features of an excluder are not available. Some of the aspects are decided by hydraulic calculations. Others are based on experience of previous works. Quite a few are adopted rather arbitrarily. It is essential that in case of any major headwork, the design of the excluder should be carefully tested by models before final adoption.

(v) Fish ladder, Navigation Lock or Free gap that may become necessary on the basis of special requirement for a particular situation.

Chapter 3

LAYOUT CRITERIA FOR DIVERSION WORKS

3.1. The layout of any diversion work is mostly governed by the type and factors governing location of its various constituents, viz. -

i) Type of diversion structure;
ii) Position of undersluices and their crest level;
iii) Position and length of divide wall - Design of of pocket;
iv) Shape of guide banks;
v) Alignment and sill level of the head regulator.

Each site has its own features some of which may fit in with in the best limitations of the above requirements and some may be to a disadvantage. A combination of these dictated by economical considerations and dependable operation would govern the criteria for layout of any diversion structure.

The various limitationson the constituents of the diversion work are as below:

3.2. Type of diversion structure

A diversion structure can be narrow or wide. Different opinions have been expressed on the type of structure to be adopted.

One school of thought (1) believes that both from experiments and observations on the prototype, it is shown that narrow and deep weirs induce more favourable conditions at the head-regulator from the point of view of sediment control. They argue that in the widerworks, formation of belas, their erosion and development of unfavourable conditions of flow are frequently experienced, which can be obviated to a marked extent in narrow and deep weirs.

The arguments in favour of a wide weir are that, since with an increase in the discharge intensity and the depth over the crest, the cost of the gates and the structure would increase and what is saved by restricting the water-way would be lost elsewhere.

According to Joglekar[2], as a result of ponding upstream, shoaling does take place upstream of a weir or barrage. A constricted water-way may remove shoals immediately upstream, but may not retard shoaling as a whole in the river upstream and also the tendency of the river to increase its tortuosity as a result of the ponding. The Sutlej Valley weirs are examples of restricted-width water-ways. Shoals persist upstream of all of them and in no case a straight, stable channel was secured and maintained.

It is obvious that the type of structure should be decided both from the point of view of sediment entry into canal and shoaling above the weir or erosion below the barrage or weir consistent with economy.

3.3. Position of undersluices and their crest level

When it is decided to construct more than one undersluices in the river, the most suitable position for locating the undersluices hasto be determined, both from the point of view of sediment control as well as its roles for the passage of flood during high river discharges. From the point of view of sediment control at a canal headworks, where still pond system is adopted a suitable position for the first set of undersluices is adjacent to the regulator especially when the approach to the pocket is along the upstream guide bank.[3] This curvature would push the main load of sediment into the second set of undersluices or to the river bays.

Second set of undersluices become necessary in case of weirs. In order to induce a favourable curvature of flow, central undersluices were specially constructed later at Khanki Headworks as right undersluices which were far removed from the left, could notwork in conjunction with the left undersluices for sediment control.

Another point for consideration in this connection is whether in case of the barrage one level of the crest for river bays as also the undersluices would be better or depressed undersluices in the barrage would be more suitable. Undersluices at level of river bays are generally unsuitable. The depressed undersluices help to maintain a well defined channel to the sluices and flushing can be done easily. It is considered that head-regulator with suitably located depressed undersluices will induce favourable conditions for sediment control into the canal.

Experiments were carried out on the model of Harike Barrage on Sutlej river for the location of the second set of undersluices. Only one canal takes off on the left side. As such, the first set was located next to the canal regulator. The second set of undersluices was examined at the right end. It was observed that the undersluices at right did not serve any useful purpose. However, as the barrage and the left undersluices were kept at the same level, the question of second set of undersluices did not arise.

If a second set of undersluices is required to be constructed, it should be so located that the two can work together to develop suitable curvature of flow.

3.4. Position and length of divide wall

A divide wall is constructed at the end of the undersluices and may be parallel or slightly splayed out to the head-regulator and extending upstream of barrage or the weir.

Divide walls have been constructed in many headworks.[3] At certain places like Emerson Barrage, there are two divide walls. At the Madhopur Headworks, a divide wall was constructed enclosing five out of twelve bays, extending upto the upstream end of the regulator. By this a narrow pocket was formed and very high velocities were generated. The divide wall got damaged and half the length was washed out. Model experiments were carried out with a divide wall covering half the length of the regulator. This divide wall did not function properly and more gravel and sediment entered the canal. The divide wall further got damaged and was later allowed to be washed away.

A divide wall for Nangal Barrage was model-tested extending from pier No. 5 (the Nangal Barrage has 26 bays of 9.3 m each), and covering the full length of the regulator. The width of the pocket formed was small and unsatisfactory conditions of flow developed. No divide wall was therefore provided. Similar conditions prevailed in Salandi Barrage. The divide wall was omitted.

Experiments were carried out for the Harike Barrage with divide-wall lengths varying from 152.5 m to 305 m. A divide wall which projected a little beyond the canal regulator gave the best results. This is contrary to the general practice of extending a divide wall upto 2/3rds of the regulator.

In most of the existing old works, a divide wall much longer than the canal head-regulator has been provided. It has now been found that a longer divide wall does not necessarily help exclusion. Moreover, the farther the nose of the divide wall from the barrage, the lesser is the effect of regulation on curvature of flow, which is of great help in getting efficient exclusion particularly at low river discharges. It also takes much longer time to scour away the silt.

Whatever may be the length determined from model experiments or otherwise, a divide wall upstream of a barrage or a weir, to isolate the canal head-regulator from the main flow, is useful in effecting sand exclusion, since it creates a quiet pool. Under the still pond regulation from which a discharge can be drawn into the canal, the improvement in exclusion resulting from a divide wall is due to imposition of a favourable curvature and the difference in discharge intensities in the pocket and in the river during high floods.

A divide wall covering 2/3rd length of the head-regulator is generally adequate when only one canal takes off from a weir or barrage. In case of more than one canal at the same side, a divide wall should extend upto the last regulator.[3]

Table 3.1 givesthe salient features of the length of canal regulator.

3.5. Shapeof guide banks

The various types of guide banks are:

a) parallel guide banks

b) coverging guide banks

c) diverging guide banks

d) bottle-neck guide banks

e) concave guide banks

f) concave-convex guide banks

The first four are generally preferred though at the Sûlemanki Headworks[2] the bottle-neck type was provided, which has not proved successful, rather it gave trouble.

The general considerations in fixing the alignment of the guide bank should be to make the best use of the river energy to develop further suitable conditions such as a deep channel along the guide bank so as to create favourable curvature. It is also important to see that water follows the guide banks at all river stages under different conditions of regulation of barrage.

Sometimes a study of the performance of the existing guide banks coupled with model experiments gives a fair approach for design of new works. This principle was followed in fixing the alignment of Harike Barrage. The design of guide banks has been discussed in Chapter 9.

Table 3.1 - Statement shirs

Sl. No.	Name of Barrage Weir	of respect r	Length of divide wall downstream of axis (m)	Length beyond launching apron (m)
1.	Narora Barrage	regula-	91.4	21.4
2.	Gandak Barrage: i) Left	ıder a	82.4	8.7
	ii) Right		82.4	8.7
3.	Kosi Barrage: i) West Kosi	cluder, regu-	95.5	21.2
	ii) East Kosi	gulator	95.5	21.2
4.	Dakpathar		91.5	19.95
5.	Ashan		84.7	9.14
6.	Ramganga	of	79.5	Nil
7.	Ferozepur: i) Left	of	-	-
	ii) Right		-	-
8.	Nangal			
9.	Rupar	end of	73.1	-
10.	Harike	end of	87.4	-
11.	Khanki		-	-
12.	Tajewala Headwork			
13.	Madhopur Headwork (Upper Bari Doab	s from a pocket on the left.		
14.	Salandi Barrage			

Table 3.1 - Statement showing length of divide walls as adopted in existing Barrages/Weirs

Sl. No.	Name of Barrage or Weir	Length of divide wall (m): Length of canal regulator	Length of divide wall (m): Distance of far end of regulator from axis	Length of divide wall (m): Length of divide wall U/S of axis	Position of Nose of divide wall with respect to Head Regulator	Length of divide wall downstream of axis (m)	Length beyond launching apron (m)
1.	Narora Barrage	63.4	67	67	Upto far end of regulator	91.4	21.4
2.	Gandak Barrage: i) Left	71.8	94.5	122	End of silt excluder and 27.4 m beyond regulator	82.4	8.7
	ii) Right	71.8	94.5	122		82.4	8.7
3.	Kosi Barrage: i) West Kosi	40.7	65.2	88.4	i) End of silt excluder, 23.2 m beyond regulator	95.5	21.2
	ii) East Kosi	98.1	122.5	122.5	ii) Upto end of regulator	95.5	21.2
4.	Dakpathar	53.5	90.0	88.5	End of regulator	91.5	19.95
5.	Ashan	39.9	77.0	79.7	End of regulator	84.7	9.14
6.	Ramganga	50	57.6	40.8	Upto 70% length of regulator	79.5	Nil
7.	Ferozepur: i) Left	64	106.8	216	106.8 far end of regulator.	-	-
	ii) Right	79.4	114.2	222		-	-
8.	Nangal	---------- No divide wal ------					
9.	Rupar	97.8	104.5	231.0	128 m beyond far end of regulator	73.1	-
10.	Harike	131.1	152.5	163.5	21.3 m beyond far end of regulator	87.4	-
11.	Khanki	198.2	-	167.8	-	-	-
12.	Tajewala Headworks, W. Yamuna Canal - No divide wall						
13.	Madhopur Headworks (Upper Bari Doab Canal)	91.5	No undersluices but advanced undersluices from a pocket on the left.				
14.	Salandi Barrage	No undersluices and no divide wall					

3.6. Alignment and sill level of the head-regulator

3.6.1. The alignment of a head-regulator plays a vital role in any headworks layout. Prevention of entry of silt into the canal should be one of the main objects in any location of a head-regulator since a considerable amount of money has to be spent later in silt clearance every year.

The angle of off-take is the most important factor in the location of a head-regulator. Different opinions have been expressed in respect of this off-take angle.

In India opinions are expressed[3] in favour of an angle of 10° to 12° though in Uttar Pradesh in most of the major structures the angle varies from 15° to 20°. A 90° off-take has also been provided in some of the existing major works, constructed in nineteenth century.

According to Schoklitch[4] an intake anle of 90° is wrong under all circumstances. Experiments were conducted by him with variously located intakes and fifferent diversion ratios (i.e. the ratio of the flow diverted,to the stream flow). A comparison of these angles revealed that there is no such thing as a correct intake angle, for this angle varies with the diversion ratio and also with a position on the intake bend. The acuteness of the intake angle increases as the diversion ratio decreases. The diversion ratio fluctuates continually withe the river discharge and the diversion. The angle varies with the diversion ratio and should be chosen tosuit the conditions existing when the bed load is high, i.e. when the diversion ratio is small; this means that the intake angle should be made acute. An angle of 90° is wrong under all circumstances.

This also applies to inlet piers; they also should make an acute angle with the river. This aspect has been taken care of at Narora where an additional tilt of 5° is given to the regulator piers.

In his experiments Schoklitsch showed that most of the bed material from parent channel entered the off-take when the angle was changed from 30°,to 150°, the discharge in the two channels being the same. Although the experiments were conducted under different conditions from those prevailing at diversion works, they nevertheless indicate plainly that the optimum results are obtained with an intake tangent to the concaves ide of the bend, so that the diversion canal is a virtual extension of the river channel, while the surplus water flows away through the bend as through a branch.

According to Leliavsky[5] a centrifugal force is engendered at the off-take point, where the water particles deviate from their normal course into the side channel. According to him the centrifugal force increases with the angle of off-take and is almost constant at 150°.

Interesting enough, experiments were conducted at Roorkee University[6] in respect of the off-take angle. The angles adopted were 30°, 60°, 90°, 120° and 150°. The percentage of sediment entering the off-take was least in case of 30° off-take when compared with other four angles. Further, the concept of existence of a centrifugal force at the off-take as put forth by Leliavsky, was found to be true which increased with the increase of off-take angle.

Table 3.2 gives the alignment of the regulators as adopted

Table 3.2 - Alignment of head-regulators in different barrages and weirs

Sl.No.	Name of Barrage or Weir	Alignment of regulator w.r.t. the line of right angle to the barrage/ weir
1 1.	Narora Barrage	17° (with an additional tilt of 5° in regulator piers).
2.	Gandak Barrage:	
	Left	0°
	Right	0°
3.	Kosi Barrage:	
	i) West Kosi	12.5°
	ii) East Kosi	12.5°
4.	Dakpathar	20°
5.	Ashan	17°
6.	Ramganga	15°
7.	Ferozepur Head works:	
	Left	15°
	Right	14°
8.	All American Canal, U.S.A.	21°
9.	Rasul Headworks	13.5°
10.	Khanki Headworks	15°
11.	Nangal Barrage	12.25°
12.	Madhopur Headworks	0°

(continued)

(continued)

13.	Rupar Headworks	15° in reverse direction
14.	Harike Barrage	11.1°
15.	Marala Headworks	0°
16.	Kalabagh Barrage	0°
17.	Panjnad Headworks	13.5°
18.	Salandi Barrage	0°

at different existing works. It can be seen that the angle of off-take varies to a maximum of 20° in Indian works contrary to Uppal's findings of 10° - 12°.

3.6.2. Another important consideration in the layout of the head regulator is the elevation of the sill. The prevalent practice in this country is to keep the sill of the canal regulator higher than the level of the floor of the pocket, and the crest of the undersluices. This is the first measure taken for sediment exclusion as it provides a margin for ramp formation.[3] On many headworks the sills of the regulators have been raised so as to allow the highly-charged bed water to be passed through the undersluices and the upper water with a low sediment charge to be let into the canal so as to reduce sediment entry into the canal. At some of the old canal headworks, the regulator sill was raised subsequently with a view to minimise sediment entry into the canal.

Joglekar[2], however, disagrees with the idea of keeping a very high sill. According to him the trap provided by higher sill will get filled in 3 to 4 days and it is not possible to clear the trap by flushing the sand as often as one would like, since such flushing would seriously interfere with the normal working of canals. Once the pocket is filled up, sand particles which roll into the pocket, find their entry into the canal regulator irrespective of height of sill above pocket level.

According to Schoklitsch[4], the elevation of the inlet sill and its position relative to the weir or barrage, are of great influence in preventing detritus from entering the intake. The widely accepted view that the height of the inlet sill above the original river bed or above the sill of the adjacent scouring sluices is the governing factor, is completely erroneous; it is rather the depth of the inlet sill below the full-reservoir level which determines its effectiveness in preventing the entrance of detritus. The diversion ratio, i.e. ratio of the flow diverted, to the stream flow, also influences the movement of detritus at the intake. The greater the diversion is, the more detritus enters the intake. The greater the ultimate diversion is to be, the higher the inlet sill should be placed.

According to Leliavsky(5), the water depth over the sill of the regulator must not exceed one-quarter or one-third of the water depth measured in front of the headworks in the river from which the supply is drawn and the velocity above the crest must not be greater than in the canal it feeds.

The above criteria, however, recommend a higher crest. But the fact remains, a higher crest results in a large water-way and consequently the structure would be costly, but this can be compensated by the saving in annual silt clearances. Table 3.3 shows the crest level of head regulators as adopted in the existing regulators. The crest in all cases is kept higher than the crest of the undersluices.

Joglekar(2) puts forth the following two points disapproving a wide water-way for head-regulator:

i) The regulator width sometimes becomes larger than the canal width downstream. Experience at Sukkur has shown that the carrying capacity of end-spans is considerably reduced under such conditions, putting, sometimes, the end-spans completely out of action.

Table 3.3 - Statement showing sill level of head-regulators and Crest level of undersluices

Sl. No.	Name of Barrage/Weir	Reduced level of sills		
		Upstream under-sluice floor	Crest of under-sluice	Crest level of regu-lator
1.	Narora Barrage	174.50	174.50	176.0
2.	Gandak Barrage:			
	i) Left	104.20	104.20	106.4
	ii) Right	104.20	104.20	106.4
3.	Kosi Barrage:			
	i) West Kosi	70.1	70.1	72.00
	ii) East Kosi	70.1	70.1	72.00
4.	Dakpathar	450	450	451.68
5.	Ashan	395.2	395.95	397.50
6.	Ramganga	223.10	223.10	225.10
7.	Ferozepur Headworks:			
	i) Left	192.5	193.41	194.78
	ii) Right	192.5	-	194.78
8.	Khanki Headworks	217.5	217.5	221.16
9.	Nangal Barrage	335.0	337.44	342.62
10.	Rupar Headworks	261.0	261.0	263.74/265.26
11.	Harike Barrage	203.0	204.67	-
12.	Salandi Barrage	35.3	No under-sluices	34.74

ii) Experiments have also shown that for effective sand exclusion from canals, the nose of the divide wall should extend upto the upstream abutment of the first canal, and should be as near the barrage as possible. With higher sill the head-regulator becomes wider, the divide-wall length increases, thus shifting the control point, i.e. the nose which when nearer the barrage is beneficial at the time when the wedge-shapre regulation is required.

3.7. Each of the features stated above has implication in fixing the various dimensions of the diversion structure as a whole and holds a criterion for layout. In actual practice, several layouts have to beworked out and the best possible is determined from model experiments.

3.8. <u>Specific problems</u>

Though it is held out that a divide wall covering two-thirds of the regulator width would be adequate for the exclusion of silt, it is interesting to note that a long divide wall was ultimately required to check the silt inflow into the Krishna canals taking off from the Krishna Barrage.

The old anicut across the river Krishna at Vijayawada was washed off in the year 1952. A new barrage consisting of 70 spans of 12.19 m each incorporating the old anicut as its end-sill was constructed in the year 1954.

After operation of the barrage it was noticed by the project authorities that large quantities of sand were drawn by the Krishna East Main Canal (left bank canal).

Experiments to improve sand exclusion from the left bank

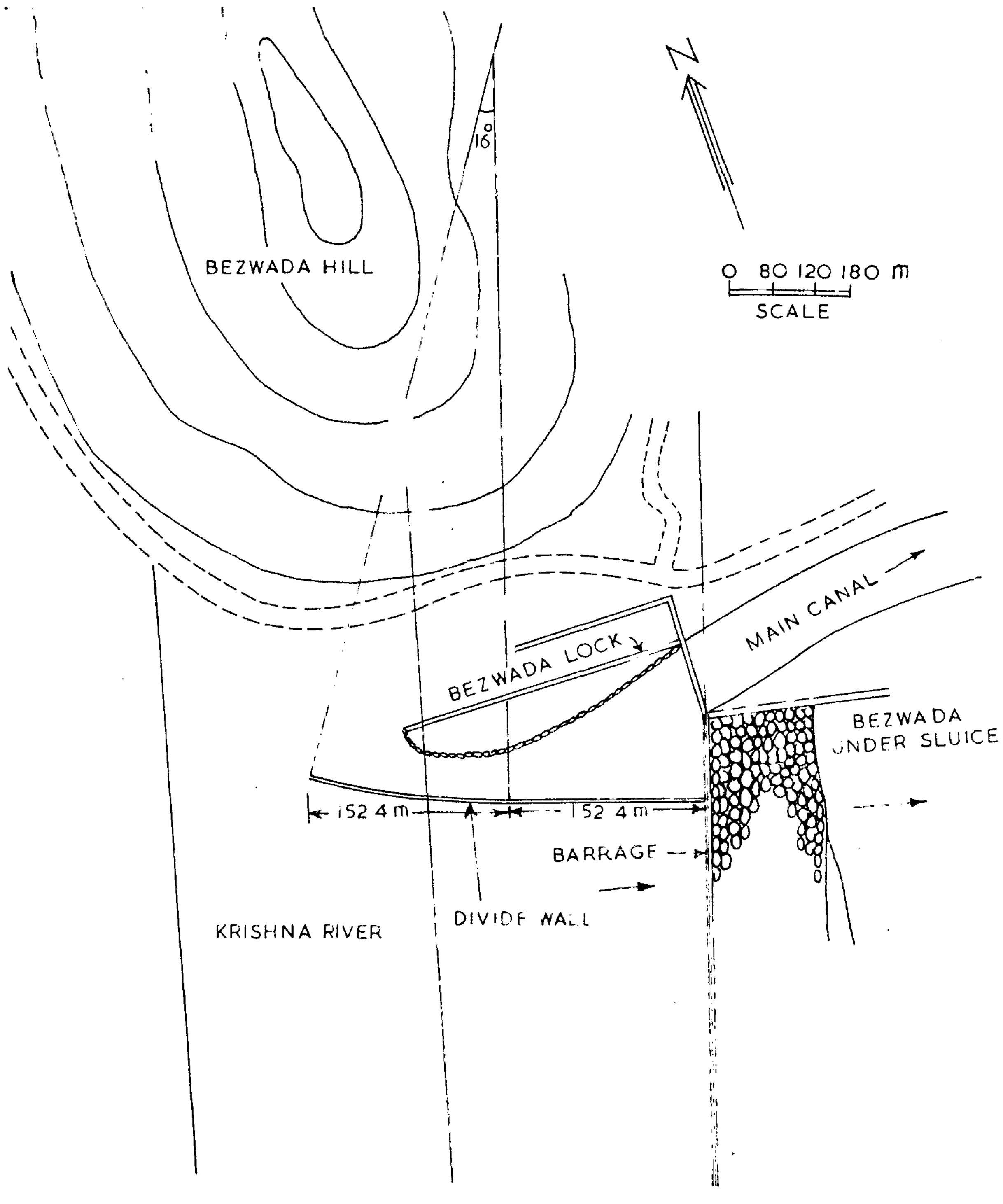


FIG. 3.1 DETAIL CURVED DIVIDE WALL OF KRISHNA BARRAGE.

canal by changing the existing regulation, were conducted at the Central Water & Power Research Station, Poona.(31)

At the site still-pond regulation with pond level at R.L. 17.37 m was followed. The undersluices were always kept closed and functioned only during the flushing operations. The discharge of the canal (L.B.) varied from 113.27 m^3/sec. to maximum 311.44 m^3/sec., and the canal was run during high floods also. Heavy silting was observed during flood stages between 2,831 m^3/sec. and 11,326 m^3/sec. The general curvature along the left bank was favourable for sand exclusion for a length of 0.8 km upstream of the barrage. Due to the hill projection near the Vijayawada side of the barrage, the curvature gets locally reversed. (Fig. 3.1). In addition, the sill of the canal head-regulator at R.L. 12.37 m was about 1.5 m lower than the crest of the main barrage, at R.L. 13.75 m, and the shape of the pocket is peculiar. The pocket is narrow at the entry, near the nose of the divide wall and widens towards the head regulator, behaving more like a settling basin. The deposited material thus raised the bed levels of the pocket, even higher than the sill of the regulator, which results in the accumulated material gradually entering the canal evenwith smaller canal discharges. The bed-building stageof the river was calculated at 5,663 m^3/sec. Hence sand exclusion was studied for discharges 5,6663 m^3/sec. and above.

Considering a longer pocket for maximum sand exclusion, the divide wall was extended by 228.60 m beyond the existing 76.20-m length, thus making a total straight length of 304.80 m. Great improvement in exclusion of sand was noticed. But it was noticed that a return flow was created on the right side of the divide wall in front of the barrage spans, adjacent to it. Hence, the approach conditions at the straight head of the divide wall did not appear to be favourable. It was decided to modify the regulation by passing higher discharges through the adjoining 10 spans of the barrage on the right of this divide wall by completely opening the gates for all river stages from 2,831 to 11,326 m^3/sec. The return flow still persisted. It was decided to provide a curve to the divide wall at its upstream end, so that the flow could hug to the right side of the divide wall. A divide wall with a curvature of radius 571.50 m towards its upstream end was found satisfactory.

The total length of the divide wall was therefore increased to 304.80 m. The modified regulation now proposed was to keep 10 spans fully open, of the main barrage adjacent to the divide wall for all stages upto 7,079 m^3/sec. and the remaining gates of the barrage opened partially and equally so as to maintain the pond level at R.L. 17.37 m. For adopting this regulation, extra energy-dissipation arrangements for increased discharge intensities, were necessary.

A study of the index plan of this barrage, however, appears that the barrage was not located properly. The canal off-takes from the concave side of the bend formed by the projection of the hill into the river. The only criteria adopted for locating the barrage appears to be, to take advantage of the existing training works of the old weir.

Further, the sill of the canal has been kept lower than the crest of the undersluices. Obviously, allthe sediment would enter into the canal. The wrong provision of the pocket with a converging divide wall also added to the silt entry into the canal.

(ii) The Tajewala Headworks

The Tajewala Headworks across Yamuna consist of undersluices pockets on both banks with canal regulators for the Eastern and Western canals. The Western Yamuna Canal pocket is located considerably downstream of the point where Eastern Yamuna Canal takes off and the two undersluices are joined by a very oblique weir in between. A series of training works extedning upto 8 km upstreamof the headworks have been constructed. The waters are to be shared both by the Uttar Pradesh and the Haryana Governments.

This is a peculiar weir of its kind where the main flow is parallel to the weir alignment contrary to all laws of hydraulics. (Fig. 3.2)

The total length of the weir after subsequent modifications has been kept equal to 565 m, the crest level varying in different reaches.

On the eastern side from where the U.P. Government draws its share of supplies the undersluices consist of 7 bays of 6.1 m each, where also the crest level in different bays varies.

On the western side from where the Haryana Government draws its supplies the undersluices comprise 18 bays with their crest at one level but of varying widths.

With this peculiar orientation both the canals are not able to get their share of supplies due to sedimentation problem, even

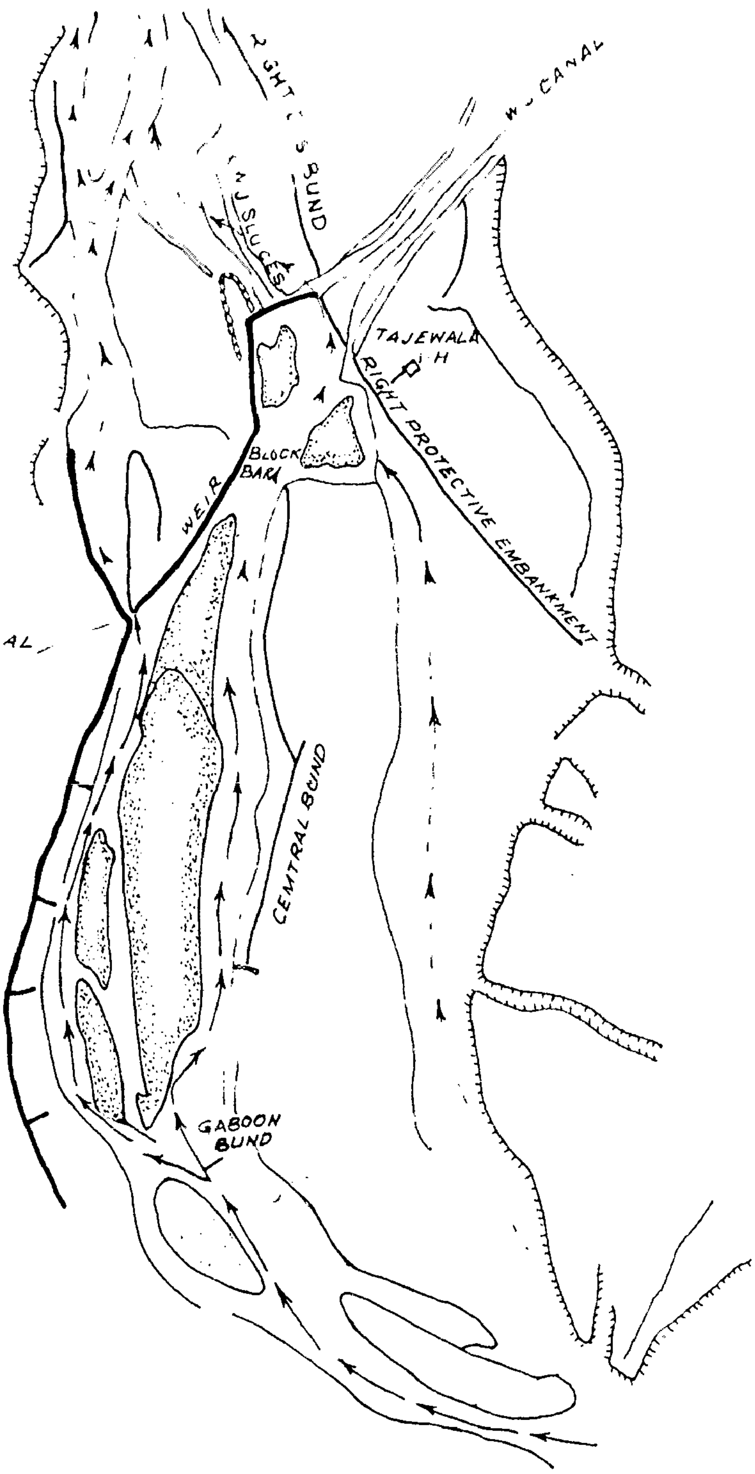


FIG. 3.2 TAJEWALA HEADWORKS ON RIVER YAMUNA.

when there is enough water available in the river.

The head-regulators are defectively sited with respect to the main flow of the Yamuna, with the result that the pockets upstream of the head-regulator remain shoaled upto 0.91 m to 1.22 m, above the crest level of the canal. The modifications carried out gave only temporary relief.

To obviate these difficulties the two Governments are left with no choiçe but, inter alia, to agree for the construction of a new barrage.

(iii) Palar Barrage

Experiments were conducted at the Poondi Research Station (39) to evolve a suitable device to exclude coarse bed load from canals of erstwhile Palar Anicut System as it was proposed to be converted into a barrage. This is a case where surplus waters for scouring operations will be negligible.

Observations on the model clearly showed that the following three conditions will have to be realised for successful functioning of the divide wall:

a) The flow should follow the bank;

b) It should get divided at the top;

c) Higher velocity should be obtained on the river side.

All the three purposes were achieved by another wall longer than the divide wall on the river side just one vent, away. This wall was specially effective in creating high velocity flow in the passage. To improve the functions, a breast wall at the entrance to the passage was incorporated. The top of the breast wall was kept at E.L. 153.5 m and the bottom at

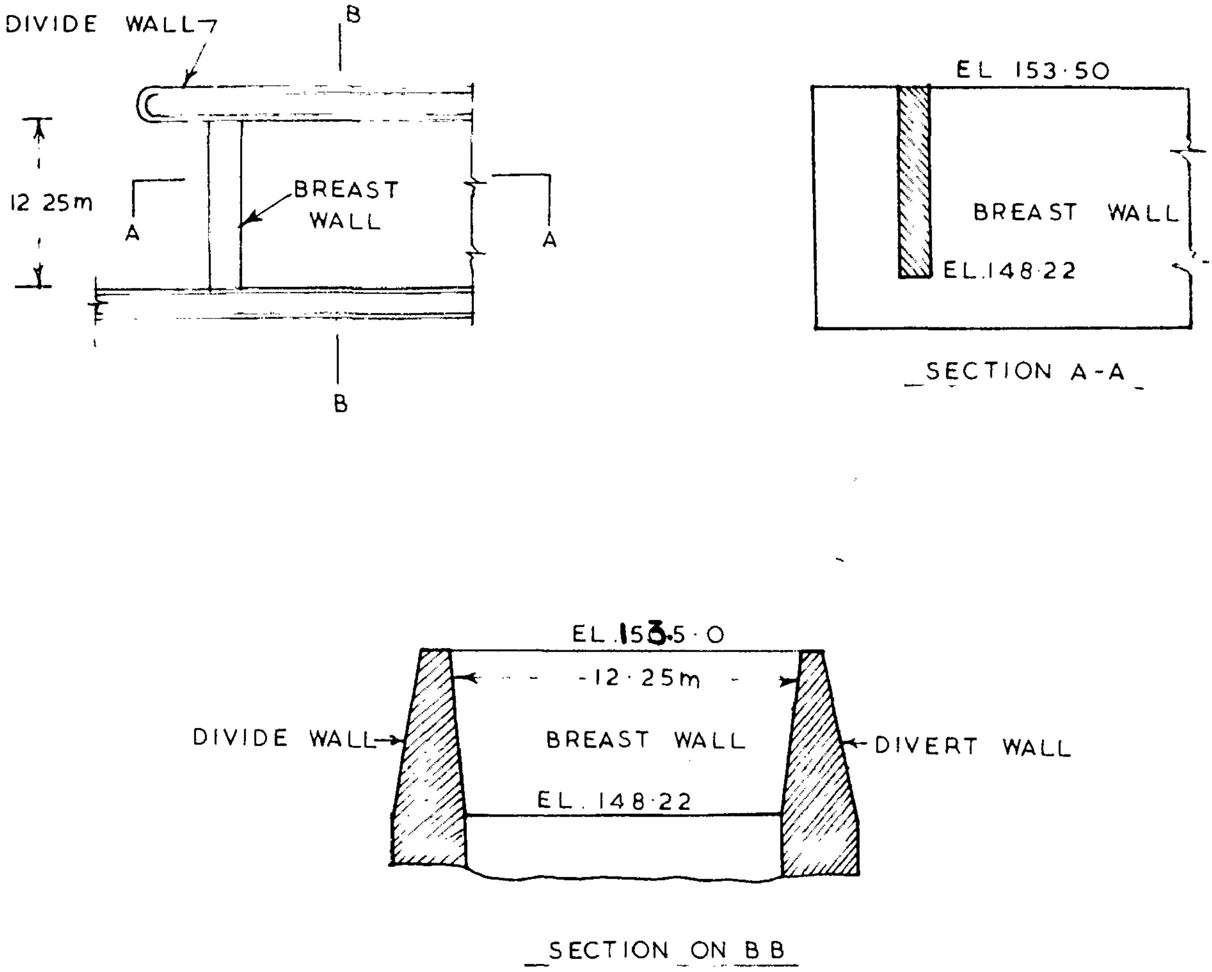


FIG.3.3 DETAILS OF DIVERT WALL ADOPTED IN PALAR BARRAGE.

148.22 E.L. (Fig. 3.3). Thus at the entry a vent of 1.0 to 1.5 m is formed, above the river bed. On an average 80% reduction of sediment entry was obtained.

This is a unique case. Obviously, higher velocities are achieved by providing restricted water-way. The flow is also diverted towards the vent way by the long divide-wall. However, severe scour may develop which has to be guarded against.

Chapter 4

EARLIER THEORIES IN WEIR DESIGN

4.1. The law of flow of water through permeable soils was enunciated for the first time in 1856 by Darcy, who, as a result of experiments, found that the velocity of flow varied directly as the head and, inversely, as the length of the path of flow. This law is expressed by the equation

$$V = K\frac{H}{L}$$

where,

V = Velocity

H = Head

L = Length of path of flow

K = A constant calledthe transmission constant

The validity of this law in relation to weir design was tested by Lt.-Col. Clibborn, Principal of the Thomason College of Engineering, Roorkee, in 1896 in connection with the proposals for repairs to the damage in 1895 to the Khanki Weir, on the Chenab River. This weir which feeds the Lower Chenab Canal, was complated in February 1892. In January 1895, 100 ft. of the weir crest in Bay No. 1 subsided about 60 cms. In order to investigate the causes of this damage and the means of ensuring future safety a series of experiments were carriedout with Khanki sand at the Thomason College, Roorkee.

These historic experiments were carried out by Lt.-Col. Clibborn with a tube 36 metres long and 60-cm internal dia. filled with Khanki sand. The relationship obtained from these experiments, between velocity, headand length of path of flow, was in keeping with that of Darcy, except that at very high heads slight departures were noticed.

The experimental results obtained by Clibborn were then checked on a prototype at Narora where there was 3.6-metre head of water. Holes were drilled at selected points in the floor to test the actual percolation pressure. On the 27th March 1898, two pipes were ready for pressure observation. This experiment showedclearly, that the upward pressure at the point had reached an intensity which reduced the stability of weir very precariously. However, unfortunately, on the 29th March, two days after the experiment, a length of about 200 ft. of the weir and 600 to 700 ft. away from the site of observation, was blown up. The river bed was protected upstream and downstream of the weir by pitching, partly grouted and partly dry. A layer of puddle clay was placed beneath the upstream protection. Shortly, before the failure a great part of this protection, and the layer of clay placed under it, were washed away by a cross-current set up by a flood, and as a consequence, the pressure of water filtering under the structure rapidly increased, so that the 15 metres deep masonry floor was burst upwards.

The failure of Narora Weir and the Khanki Weir gave great prominence to the subject of percolation water pressures discussed by Col. Clibborn in his note. These ideas later originated Hydraulic gradient theory for weir design, apparently, between Sir JohnOttley and Thomas Higham. With the publication of the results of Col. Clibborn's experiments in 1902, the Hydraulic gradient theory came to be generally accepted in India.

4.2. Bligh's Theory

In 1907, Bligh, in his book on ''Practical Design of Irrigation Works'' believed rather that the stability of a weir depended on its weight.[7] But in the 1910 edition of his book he admitted the fallacy of his original belief and became converted to the ''Hydraulic gradient theory'' of Ottley etc. Bligh's enunciation of this theorywas later universally accepted as Bligh's creep theory.

In this theory, Bligh assumes as an approximation, that the hydraulic slope or gradient is constant throughout the length of creep (a b c d). It follows, therefore, that the velocity of filtration, which must be proportional to the gradient is also constant. Thus the gradient diagram is represented by a triangle, the base of which is equal to the length of creep, a b c d. (Fig.4·1) This length is termed'the creep' and usually denoted by the letter L. It is meant to represent the length of the path followed by a filtering particle of water. Bligh believes that the apron is safe against undermining if the ratio $\frac{L}{H} = C$ is not less than the safe value assigned to it for the given class of soil. The values recommended by Bligh are:

Class I: River beds of light silt and sand, of which 60% passes a 100-mesh sieve, as those of the Nile or the Missisippi, C = 18.

Class II: Fine micaceous sand of which 80% of the grains pass a 75-mesh sieve as in the Himalayan rivers and the Colorado, C = 15.

Class III: Coarse-grained sands, as in the central and south India, C = 12.

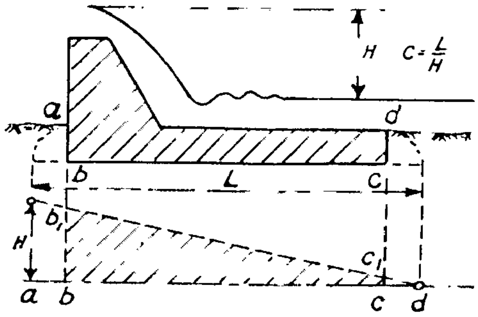


FIG. 4.1

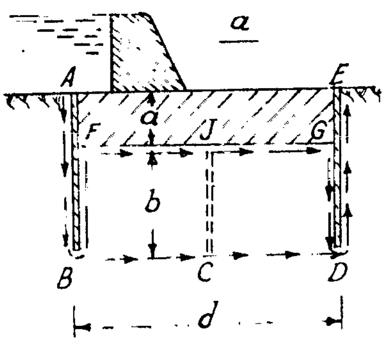


FIG. 4.2

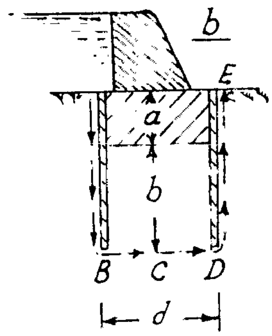


FIG. 4.3

Class IV: Boulders or shingle and gravel mixed with sand, C varies from 9 to 5.

The second condition of equilibrium in Bligh's theory is that the weight of the apron must be sufficient to counterbalance the uplift pressure.

Bligh stated that the length of flow had the same effectiveness, length for length, in reducing uplift pressures, whether it was along the horizontal or the vertical. Herein lies the danger of applying Bligh's otherwise simple formula. Hence, according to Bligh the percolation flow instead of following the short-cut indicated in Fig. 2 by A B C D E, ''clings to'' or ''hugs'' the line of contact between the solid work and permeable soil, shown ABFJGDE. Thus the length of creep is not $2a + 2b + d$ but is equal to $2a + 4b + d$.

If H = the total head over the weir; the loss of head per unit length of creep would be

$$\frac{1}{C} = \frac{H}{2a + 4b + d}$$

The loss of head per unit length, or what is the same thing as the average hydraulic gradient (1/C), he called 'C' the percolation coefficient.

Bligh also fixed a limit for application of his assumptions. In case of sheet piles driven too close a distance from one another the method might not be applicable. He, therefore, stated that his method holds good so long as the horizontal distance between the pile lines 'd' was greater than twice their depth, 'b'. On the other hand, the line of creep followed the path shown in Fig.4·3.

Because of its simplicity, this theory found general acceptance.[7] Some works designed on this theory failed while others stood, depending on the extent to which they ignored or took note of the importance of vertical cutoffs at the upstream and downstream ends. The only experimental data available at the time of Bligh's theory were those of Col. Clibborn, and perhaps the only field observations those at the Narora Weir. From these meagre data Bligh evolved a simple formula, which, however, fitted neither the Clibborn results with sheet piles nor those at the Narora Weir - and recommended it for general use to the profession. Furthermore, the hydraulic gradient is assumed to be constant over the entire length of the line of creep, though actually the new seepage theory as now developed shows the gradient varying widely at different points of the seepage path. The assumption that the flow occurs along the lines of contact is also not correct.

4.3. Lane's creep theory

A fairly comprehensive summary of failures with an analysis of creep ratios has been given by Lane.[7]

Professor Lane's approach differs from all other solutions of the percolation problem, wherein he aims at a new criterion, derived from the ''line of creep'' concept first suggested by Bligh. He, however, states that Bligh's statement that the water follows the line of creep and not the path of least resistance, is in error. According to Professor Lane, the water-way occasionally travels along the line of creep, because it is difficult to secure intimate contact between the flat surface of the solid foundation of a dam and the granular soil upon which it rests. If there is poor contact, then water percolating along the line of creep will meet with less resistance than that which travels through undisturbed soil. This will then be the most dangerous point in the entire width of the dam, where the highest percolation velocity may be expected to take place and failure is, therefore, the most probable.

In actual practice the contact between earth and deep sheet piles is more likely to be intimate than for concrete foundation cast over a flat bedding. This tends to suggest that in calculating the length of creep, one should discriminate between vertical and horizontal surface, greater ''weight'' being attributed to the former than to the latter. Hence the concept of ''weighted creep''. Also in the frequently occurring stratified soil formations, vertical cutoffs stop the flow through the weak layers, and force the water to percolate through the less permeable strata, or to follow a longer route; which means that here again the vertical obstructions are more effective than a corresponding length of horizontal creep.

Lane had examined 278 dams and weirs of different descriptions built on various soils. With these data he established the principle of weighted creep theory. In order to find the true ratio of the respective 'weights' Lane chose the one-to-three ratio, as being the value which best suited the available information on the numerous dams he examined.

If N be the sum of all the horizontal contacts and of all sloping contacts less than 45° (to the horizontal); also, let V represent the total sumof vertical contacts plus the sloping contacts greather than 45°; the weighted creep will then be

$$L_w = \frac{1}{3} N + V$$

To ensure safety against piping L_w must not be less than C_1H, where H is the total head, i.e. the difference between upstream and downstream levels, while C_1 is an empirical coefficient depending on the nature of the soil in the foundation. These values vary from 8.5 for very fine sand or silt, to 1.6 for very hard clay or hard pan.

Lane's theory is an empirical approach based on experience. The application of the theory can only be attempted in design, provided the full limitations of the same are clearly understood. The factor of safety based on this design would be uncertain.

4.4. Pavlovsky's theory

Pavlovsky approached the problem of the flow of water through sub-soils of hydraulic structures from the analogy of flow of electricity through a conductor. According to Ohm's Law -

$$\text{Current} = \frac{\text{Potential difference}}{\text{Resistance}} = E / \frac{\rho L}{A}$$

where,

E = Potential difference

A = Area of cross-section

L = Length of conductor

ρ = Sp. resistance

This is identical with Darcy's equation for flow of water through sand, viz.

$$V = K \frac{H}{L}$$

The work was published in Russian. Pavlovsky achieved success in solving a number of problems, but as the Laboratory results could not be shown to agree with field results[8], this method did not inspire confidence among the engineers and remained more or less of academic interest.

4.5. Dr. Khosla's work

In 1926-27 trouble at the syphons under the Upper Chenab Canal became acute. Cracks appeared atthe upstream and downstream ends due to undermining of sub-soil. Repairs were carried out on the accepted Bligh theory but the trouble persisted. Investigations on these led to the following conclusions:[8]

a) The outer faces of the end sheet piles were much more effective than the inner ones and the horizontal length of floor;

b) The intermediate piles if smaller in length than the outer one, were ineffective except for local distribution of pressures;

c) Undermining of floors started from the tail end. If the hydraulic gradient at exi$t was more than the critical gradient for the particular sub-soil, the soil particles would move with the flow of water, thus causing progressive degradation of the sub-soil, resulting in cavities and ultimate failure;

d) It was absolutely essential to have a reasonably deep vertical cutoff at the downstream end to prevent undermining.

Also with further investigations at these syphon sites, in 1928-29, Dr. Khosla came to the following conclusions:

a) The flow of water through the sub-soil is in stream lines and therefore susceptible to mathematical treatment;

b) The ratio (ϕ) of uplift pressure (P), at any point along the base of a particular weir founded on permeable soil, to the total head (H) is constant and independent of

i) Head (H)

ii) Class of sub-soil so long as it is homogeneous

iii) Upstream and downstream water levels.

4.6. Theory of seepage flow

For a homogeneous soil which obeys Darcy's law, the conditions of steady seepage in a two-dimensional plane can be expressed by the Laplace equation -

$$\frac{d^2\phi}{dx^2} + \frac{d^2\phi}{dz^2} = 0$$

This differential equation governs the distribution of the 'flow potential' $\phi = kn$, where k is the coefficient of permeability of the soil as defined by Darcy's Law and n is the head at any point within the soil. The solution gives two sets of curves known as 'Equipotential Lines' and 'Stream Lines' (or flow lines), mutually orthogonal to each other.(fig 4·4)

The path along which the individual particles of water seep through the soil is represented by the stream lines. The first stream line follows the outline of the base of the work and is the same as Bligh's path of cree. Other stream lines follow smooth curves providing a gradual transition from the outline of the foundation to a semi-ellipse, if the pervious soil medium extends to a very large depth. In case there is an impervious boundary at a certain depth the last stream-line will follow the impervious boundary and the intermediate stream-lines will represent a smooth transition from the first stream-line to the last.

The equipotential lines represent contours of equal head. If the downstream bed is treated as the datum, then every stream line has a head h_1 while entering the soil. As it emerges into the atmosphere its head is zero at the downstream end. Thus the head h_1 is entirely lost through the passage of the stream-line through the sub-soil and at every intermediate point in its path it has a certain residual head, h, still to be dissipated, in the remaining length to be traversed to the downstream end. Since this applies to every stream-line, it follows that there will be points on different stream-lines having the same residual head, h. If such points are joined together, the curve obtained is called an 'equipotential line.' If we assume that the downstream beds are horizontal, every particle enters with the same head, h_1; hence, the upstream surface of entry AB is the first equipotential line having a constant value of $h = h_1$. Similarly, the downstream surface CD is the last equipotential line having $h = 0$, provided no water is standing on it. In between, several equipotential lines can be drawn for values between 0 to h_1. If a number of piezometers were installed on the same equipotential line, the water will rise in all of them to the same level, as the sum of pressure + position heads is constant all along the equipotential line.

The combination of the stream lines and equipotential lines is called the 'flow net'. Once the flownet for a given problem is obtained all the effects of seepage can be easily computed from it. The distribution of uplift pressures on the base is determined by the intersection of the equipotential lines with

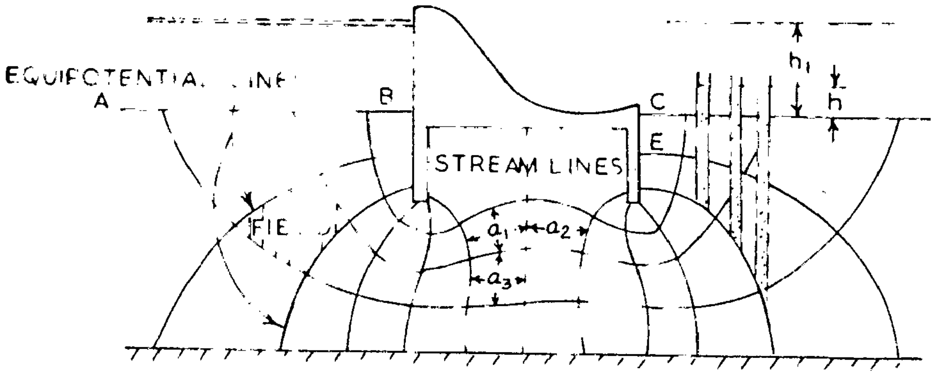


FIG. 4·4 FLOWNET .

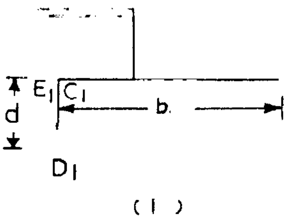


FIG. 4.5

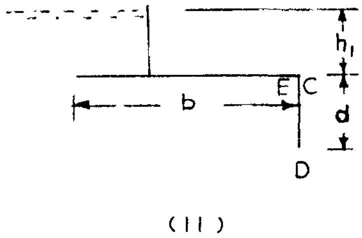


FIG. 4.6

SHEET PILE AT ENDS

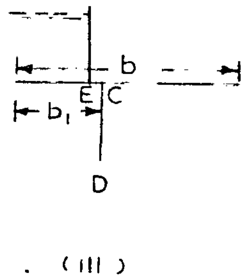


FIG. 4.7 SHEET PILE AT INTERMEDIATE POSITION.

the base.

There are a number of methods of obtaining flownets, viz. -

i) graphical sketching

ii) the electrical-analogy method

iii) the Relaxation method

iv) Dr. Khosla's method of independent variables;

etc.

For hydraulic structures, Dr. Khosla's method is generally used, as it is simple, quick and accurate.

4.7. Khosla's Theory for determination of uplift pressures

If Laplace equation is integrated for the given set of boundary conditions, mathematical solution of the flownet would be obtained for those conditions. This equation, however, is not amenable to a direct mathematical integration under the complex boundary conditions, particularly the base of the foundations, presented by an actual work. The principle of the method of independent variables evolved by Dr. A. N. Khosla consists of breaking up a complex profile into a number of simple profiles each of which is independently amenable to mathematical treatment. Some of these are -

i) A straight horizontal floor of negligible thickness, with a sheet-pile line at either end (Fig. 45&46);

ii) A straight horizontal floor of negligible thickness with a sheet-pile line at some intermediate position (Fig.47).

The results of the mathematical solutions of these forms are presented in the form of curves from which the percentage pressures at key points can be determined. The percentage pressures observed from the curves for the simple forms into which the profile is broken is demonstrated to hold for the assembled profile as a whole.

Chapter 5

FACTORS TO BE DECIDED IN THE DESIGN OF DIVERSION WORKS

5.1. For designing a diversion structure on permeable foundations, certain data is requird to be known, certain items are to be decided on economic and general considerations and the rest are then designed on the basis of these considerations. The data required to be known is -

a) Maximum flood discharge for the river at the weir or barrage site;

b) Maximum flood levels at and near the barrage or weir site;

c) River cross-section;

d) The stage discharge curve for the river at the site;

e) The sub-soil particulars.

All this information can be obtained from topographical and hydrological surveys.

5.2. <u>The factors to be decided</u>

The factors to be decided in the design of a diversion structure are -

a) What should be the permissible afflux?

b) What should be the pond level?

c) How much water-way should be allowed for the diversion structure?

d) At what level should the crest of the barrage and undersluices be fixed?

e) How much retrogression should be allowed?

f) Type of regulation to be adopted?

5.2.1. Permissible afflux

By afflux is meant the rise in maximum flood level of the river upstream of the structure as a result of obstruction. This afflux though confined in the beginning, to a short length of the river above the barrage or weir, may extend gradually very far, till the final slope of the barrage is much the same as it was before the construction of the structure. This obviously means re-establishment of the regime by deposition of silt etc. In the diversion structures founded on alluvial sands, the afflux varies from 0.6 m to 1.2 m, more commonly 1.0 m. In very steep reaches of the rivers with boulders or rock bed, the afflux may safely be higher.

The amount of afflux will determine the top levels of guide banks and their lengths, and the top levels and section of flood protection bunds. It will govern the dynamic action downstream of the work as well as the depth and location of hydraulic jump. Joglekar(2) advocates that there is considerable advantage if enough afflux can be allowed to create jump conditions, even at the maximum flood stage, thus making the barrage modular at all stages. It makes the design more definite and gives better control in working. It also helps the operation of excluders. The structure can be narrowed.

But with a narrow weir the cost of training works will increase. The discharge per metre run, the depth of scour and, therefore, the section of the loose protections upstream and downstream as well as the depths of piples at either end will also increase. Also a larger discharge intensity involves greater risk of outflanking. Table 5.1 shows the afflux provided in various barrages and weirs constructed in the recent past and under construction in the country.

5.2.2. Pond level

The pond level is the water level required in the under-sluice pocket upstream of the canal head-regulator to feed the canal with its full supply. The full-supply level of the canal at the head depends on the levels of the area which it has to irrigate and the slope of the canal. The F.S.L. of the canal at the head will be fixed on the L-section of the canal. The pond level should be fixed considering future extension of the irrigable area and minimum working head for the head-regulator which can be 10 to 15 cm.

5.2.3. Waterway

5.2.3.1. A likely figure adopted for the water-way of a weir or barrage is the Lacey's minimum stable width for the maximum flood-discharge given by[8]

$$P_w = 4.75\sqrt{\bar{Q}}$$

where,

P_w = minimum stable width in metres

Q = maximum flood discharge in cumecs.

Lacey has correlated stable widths of rivers and canals, with discharges over a wide range and has found the above general equation. In the case of large rivers the wetted perimetreis practically equal to the surface width. The Lacey formula[8],

Table 5.1 - Afflux, water-way and Looseness/Tightness factor as provided in some of the existing Barrages/Weirs

Name of the Barrage	Narora	Dakpathar	Yamuna (below C.P.H.) (CW&PC)	Wazira-bad	Sarda	Durgapur	Kosi	Sone	Gandak	Nangal	Trisuli	Salandi	Phika	Kemri
Name of the River	Ganga	Yamuna	Yamuna	Yamuna	Sarda	Damodar	Kosi	Sone	Gandak	Sutlej	Trisuli	Salandi	Phika	Pilakhar
Afflux (m)	0.914	1.01	0.076	0.12	1.371	0.914	2.286	1.213	0.67	-	0.975	0.91	1.78	0.244
Looseness or Tightness factor B/P_w	$\frac{922.5}{575.76}$ = 1.6	$\frac{517}{582.17}$ = 0.89	$\frac{531.6}{445}$ = 1.19	$\frac{454.09}{406.29}$ = 1.12	$\frac{667}{627.83}$ = 1.06	$\frac{692.2}{603.50}$ = 1.15	$\frac{1149.14}{792.48}$ = 1.45	$\frac{1412.16}{973.83}$ = 1.445	$\frac{742}{749}$ = 0.98	$\frac{304}{456}$ = 0.67	$\frac{142.65}{313.94}$ = 0.453		$\frac{108.0}{156.66}$ = 0.69	$\frac{162.8}{181.96}$ = 0.894
Maximum Discharge -														
i) Weir	11,882	10,359	6,940	3,540	14,725	10,307	20,162	32,564	15,183	922	3,700	424	691	980
ii) Under-sluice	2,676	4,189	1,557	2,169	2,265	R. 2,525 L. 2,690	6,739	8,891	8,892	-	543	-	357	436
Total	14,558	14,548	8,497	5,709	16,990	15,522	26,901	41,455	24,075	922	4,243	424	1,048	1,416
% Discharge through undersluices	18.3	28.4	18.35	37.9	13.35	33.6	25.0	21.5	36.8	-	12.85	-	38.0	30.8

though supported by some factual data, is sometimes found not to conform with the actual flood widths of rivers, the divergence being due to factors as yet unaccounted for.

Further, this formula applies to regime channels, whose bed and slope profile must conform to a semi-elliptical section[13] though Leliavsky[9] has stated that no explicity quantitative evidence was, however, supplied to supportthis statement. Nevertheless, the formula is a good guide for determining the waterway, and, thought no directly applicable, it is useful in assessing the divergence to be allowed for fixing the exact water-way. There was a tendency to provide clear water-way from 10 to 25% more than Lacey's regime perimeter. Such weirs were termed as loose weirs. However, now sometimes watery-way less than Lacey's regime perimetre is provided.

The effects of a narrow water-way and wide water-way with regard to shoal formation have already been discussed in Chapter 4. The effects of increased intensity of discharge on the structure have been discussed under 'afflux'. It is obvious that the limit placed on the afflux also limits the minimum water-way. According to Joglekar[2], experience remains the best Guide for water-way. From the study of some barrages he recommends a limit of 28 cumecs for the intensity of discharge for fixation of water-way. This limit, however, is no guide since the value has been far exceeded in some of recent barrages constmucted in the country, viz. Dakpathar - 38.2 cumecs/metre; Sarda - 33.2 cumecs/metre; Durgapur - 37.7 cumecs/metre; Kosi - 36.9 cumecs/metre; Sone - 40.6 cumecs/metre; Gandak - 40.5 cumecs/metre. The intensity of discharge should therefore depend on the bed material through which the river flows. In otherr words, Lacey's formula can be applied to plains only where the Lacey's silt factor, which governs the scour, is quite less - around unity. For boulder reaches the Lacey's water-way would be less and hence the formula may not be applicable.

The water-way for a barrage is determined by the formula -

$$Q = C.L \left|(h + ha)^{3/2} - ha^{3/2}\right|$$

where Q is the discharge passing through the barrage or weir,

C = Coefficient of discharge

L = Length of waterway

h = Water head upstream over the crest

ha = Head due to velocity of approach

Piers are needed to form the sides of the gates in a gate-controlled way.[17] The effect of the piers is to contract the flow and, hence, to alter the effective crest length of the spillway. The effective length of one bay of a gated spillway may be expressed as

$$L = L_o - K\ N\ H_e$$

where,

L_o = Clear span of the gate bay, between piers

K = The pier contraction coefficient

N = No. of side contractions, equal to two for each gate bay

and H_e = Total head over the crest including the velocity head.

The pier contraction coeff. varies mainly with the shape and position of the pier nose, the head condition, the approach depth of flow, and the operation of the adjacent gates. The approximate K values range from 0.1 for thick blunt noses, to 0.04 for thin or pointed noses and is 0.035 for round noses. These values apply to piers having a thickness equal to about 1/3rd the head on the crest when all gates are open. When one gate is open and the adjacent gates are closed, these values become roughly 2.5 times larger.

Inpractice, for most of the barrages, the end contraction coefficient is assumed as 0.10.

In effect, the water-way vis-a-vis the crest level is determined frist without assuming any end contractions. Once these dimensions are fixed, the adequacy of the water-say needs to be checked for the maximum flood, by accounting for the end contractions.

In applying this formula, two important parameters which need consideration, are -

i) Coefficient of discharge

ii) Velocity of approach

Of these,the coefficient of discharge is of paramount importance since it affects the discharge considerably. The coefficient of discharge is best determined by model experiments for all stages of flow.

5.2.3.2. Interesting enough, Leliavsky[10] gives a criterion for the fixation of the water-way from hydraulic jump considerations. If the post-jump depth (h_2) is equal to the natural downstream water depth of the river (hr), the jump will occur on the solid apron. In order that the standing wave should not repel from the apron, the discharge must not be allowed to exceed that given by the limit $h_2 = hr$.

The discharge calculated for the hydraulic conditions of this case is, therefore, in itself a characteristic criterion for the design. Pavlovsky describes it as the ''modulus'' of a weir or barrage. It is calculated from the intensity-critical depth relation

$$q_a = \sqrt{\frac{g}{\alpha}} \cdot h_{cr}^{3/2}$$

where α is a factor introduced to compensate for the error consequent on the use of the mean velocity instead of the true velocity. α is approximately equal to 1.03.

Hence in the metric units $q_a = 3\ h_{cr}^{3/2}$. The value q_a is the maximum discharge that can be passed, per unit length of weir, or barrage, without driving the standing wave into the un-protected downstream channel, and thus creating potentially dangerous scour conditions.

Hence, if the total discharge of the river is Qr, the required effective length of the structure, i.e. the length of the overfall, or the sum of the spans of the vents, is

$$W = \frac{Q_r}{q_a}$$

However, none of the above formulae or criteria are sometimes applicable for the determination of the water-way. It has to be determined from some other considerations. The Narora and the Salandi barrages are examples of this.

5.2.3.3. The Narora Barrage is constructed across river Ganga. The barrage has been designed for maximum discharge of 14,150 cumecs. Various studies were made for different overall lengths of the barrage between the abutments varying from about 610 m to 914 m, to find out the corresponding crest levels, and cistern levels for various figures of afflux.[11] (fig 5.1)

For barrage length of 610 m, which is Lacey's water-way, the intensity of flow for design discharge was 23.1 cumecs per metre run between the abutments. The intensity between the piers would in such case be more than 27.8 cumecs per metre run for weir bays and more than 32.1 cumecs for undersluices. If formation of hydraulic jump were no consideration, the design flood could be passed with a minimum afflux of 0.91 m. In this case, however, no hydraulic jump will form for discharges upto 10,020 cumecs with concentration of flow and retrogression of bed, and upto 13,200 cumecs without any concentration of flow or retrogression of bed. Hydraulic jump will only form at design flood and high discharges more than 10,020 cumces or 13,200 cumces, as the case may be. For low discharges passing below the gates, there will be shooting flow and the jump will be formed, if at all, beyond the floor. In case of high discharges till the jump is formed, the flow will remain submerged and will be turbulent. Heavy damage might result to the floor, as also to river bed and banks, during floods. A crest level of R.L. 176.0 for weir bays is necessary for formation of jump at all discharges. The afflux in such case, however, will be 2.135 m. Although the barrage of 610-m length is desirable from consideration of Lacey's formula, the intensity of flow per metre run and the value of afflux are high and its connection with left marginal bund did not fit with the existing elaborate river training works for the old weir. Thelength of the barrage, therefore, was not kept at 610 m.

In case of barrage length of 755 m, the intensity of flow between abutments for design discharge was 18.6 cumecs per metre, corresponding intensity between the piers would be 23.2 cumecs/metrerun for the weir bays, and slightly over 27.8 cumecs/m for undersluices. The design flood could be passed with a minimum afflux of 0.794, with crest level in weir bays at 174.8. In this case, however, no hydraulic jump will be formed for discharges upto 10,500 cumecs with concentration of flow and retrogression of river bed upto 12,750 cumecs without any concentration of flow and retrogression of river bed. For low discharges passing underneaththe gates, there will be shooting flow and the jump will be formed,if at all, beyond the floor. In case of high discharges, till the jump is formed flow will remain submerged and will be turbulent. A crest level of 176.0 for weirbays is necessary for formation of jump at all discharges. The afflux in such case would, however, be 1.525 m. The intensity of flow may be acceptable, but afflux is high in this case also. Moreover, connection of 755 m long barrage also on the left did not fit with the existing works. The total length of the barrage limited to 755 m was also not accepted.

For barrage of 922-m length, the intensity of flow per metre run between abutments for design flood was only 15.4 cumecs/m and intensity between piers in such barrage is within 18.3 cumecs/m

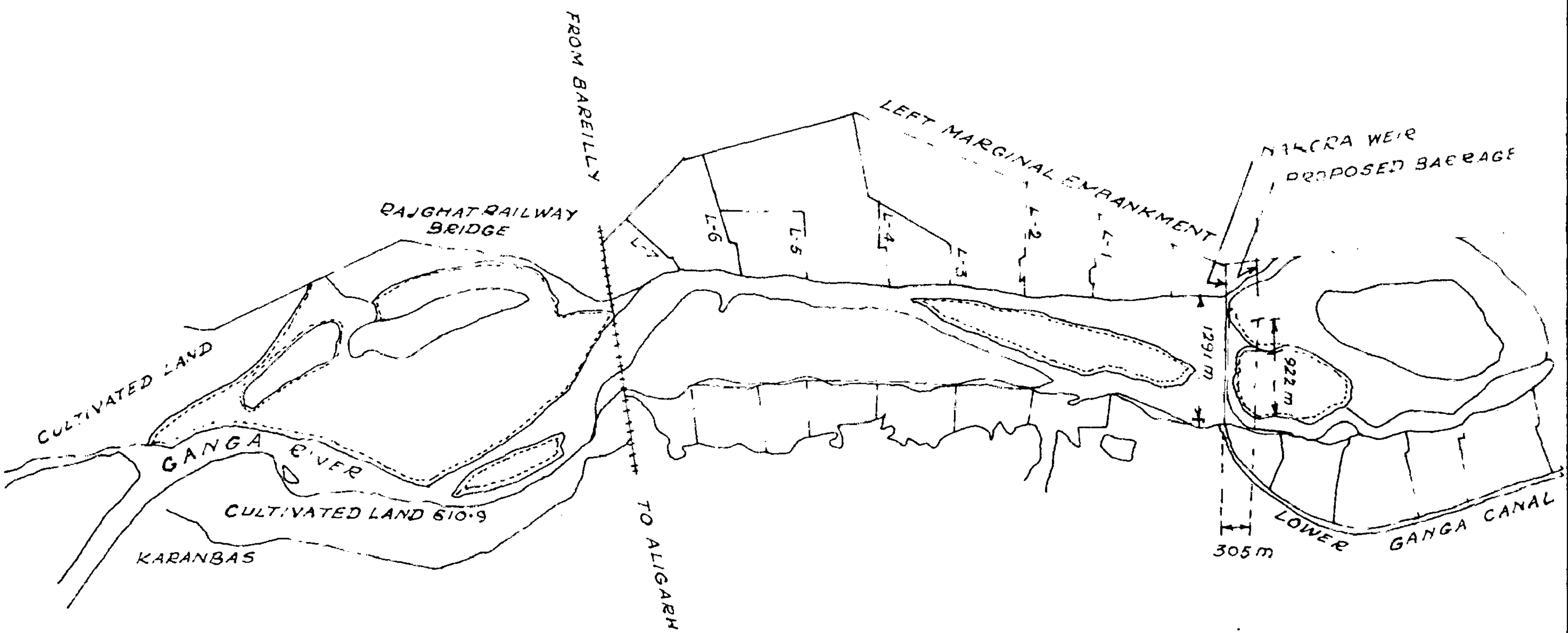


FIG. 5.1 NARORA BARRAGE INDEX MAP.

for weir bays and 27.9 cumecs/m for undersluices. The design flood can be passed with a minimum afflux of 0.62 m with crest level in weir bays at R.L. 172.8. In this case, however, no hydraulic jump will be formed between 5,300 cumecs and 8,485 cumecs, with concentration of flow and retrogression of river bed upto 12,500 cumecs, without concentration of flow and retrogression of river bed. A crest level of R.L. 176.5 for weir bays is necessary for formation of jump at all discharges. Afflux in this case will be 0.62. The connection of upstream guide bund on the left flank of the barrage with 922-m length (3020 ft) will be made with the existing marginal bund on the left through the left dividing groyne and sput No. 8. Hence, a 922 m long barrage with a looseness factor of 1.65 (Lacey's stable water-way for 14,150 cumecs equals 556 m) for design flood was adopted from all considerations.

The above experiments indicate how a limit placed on the afflux also governs the minimum water-way. Themain criterion that prevailed here for the fixation of water-way is the criterion of left guide bund with the abutment to utilise the existing river training works. The Salandi Barrage is an example of a tight barrage. The water-way for the barrage is determined as below.

The Salandi Barrage is located 8 km below the Salandi dam across the river Salandi. At the barrage site the river has a defined width of 91.5 m between its banks. Lacey's water-way for a maximum discharge of 3,250 cumecs works out to 273.5 m, i.e. 3 times the actual river width. Further, a clay stratum exists at this site below R.L. 27.4. As it is not desirable to take the sheet piles into the clay stratum from uplift considerations, the intensity of discharge, therefore, has to be limited to avoid deep scour depths. A water-way of 273.5 m would be safe from these considerations, but it is presumed that such a water-way with a high crest would lead to upstream shoal formation, and, therefore, a perpetual problem for maintenance. Studies have revealed that a clear water-way of 183 m suited all conditions. The downstream sheet pile can be taken to R.L. 28.7, i.e. 1.3 m above the clay stratum. This gives a tightness factor of 0.74.

The crest levels and the water-way for the Salandi Barrage have been fixed in the following way:

Maximum discharge	1,15,000 cusecs (3,260 cumecs)
Pond Level	132.00′(40.20 m)
H.F.L.	129.00 ft (39.3 m)
Afflux	3.0 ft. (0.915 m)
Clear span	60 ft. (18.3 m)

i) The upstream floor level has been kept at R.L. 110.00(ft) considering the deepest bed level;

ii) If two bays are to be provided for the undersluices the discharge through the undersluices will be

$$Q = 3.09 \times 120 \times (132.96-110)^{3/2}$$

$$= 3.09 \times 120 \times (22.96)^{3/2}$$

$$= 370 \times 109.50 \qquad = 40,600 \text{ cusecs}$$

Intensity of discharge through the undersluices

$$= 338 \text{ cusecs.}$$

With a silt factor of 1.0 the depth of scour will be 43 ft. and the bottom of the sheet pile shall be at R.L. 85.00, i.e. it penetrates into the clay stratum at R.L. 90.00.

Hence it was decided to omit the undersluices for the fact that the silt mostly would be deposited in the dam itself.

iii) If Lacey's water-way is provided for the barrage

$$1,15,000 = 3.1 \times 900 \times (H)^{3/2}$$

$$H = \left| \frac{1,15,000}{3 \times 900} \right|^{2/3} = 12.25 \text{ ft}$$

R.L. of crest = 132.96-12.25

= 120.71 ft.

The river bed is at R.L. 110.00, i.e. the upstream floor level. It was apprehended that a raised crest of 10.71 ft. may result in loss of control of the river and so also the huge water-way.

iv) If 600-ft. water-way is provided, i.e. 10 spans of 60 ft. -

Intensity of discharge over the crest

$$\frac{1,15,000}{600} = 192 \text{ cusecs}$$

Drowning ratio

$$= \frac{129-116}{132-116} \times 100 = 81.2\%$$

Cd = 2.96

$$\text{Head over crest} = \left| \frac{192}{2.96} \right|^{2/3} = 16.1 \text{ ft.}$$

R.L. of the crest = 132.96-16.10

= 116.86

.../

The crest level was kept at R.L. 116.00. The extra water-way ~~of~~ is 370 ft. over the natural water-way of 300 ft. The tightness factor works out to 0.74.

v) The scour depth for this intensity of discharge with 20% concentration (i.e. q = 230 cusecs), works out to 34 ft. and the sheet pile bottom would be at R.L. 94.00, i.e. 4 ft. above the clay stratum.

vi) The adequacy of the water-way with end contractions can now be checked as below:

Head of the crest = 132.96-116.00 = 16.96 ft.

$$Q = 2.96 \times (600-18 \times 0.1 \times 16.96) \times (16.96)^{3/2}$$
$$= 1{,}17{,}000 \text{ cusecs}$$

5.2.3. . An example of wrong provision of excess water-way and its effect can be best cited from the trouble faced at the Dakpathar Barrage. The Dakpathar Barrage across the Yamuna feeds the power channel on its left. The barrage has been designed for a maximum flood discharge of 14,548 cumecs. The water-way consists of 22 Nos. of weir bays 18.3 m each and 6 undersluices bays of 18.3 m each. The barrage is constructed in a boulder reach with a tightness factor of 0.89. The pond level has been kept at H.F.L. The river brings lot of debris, boulders and shingle. The regulation of the barrage is such, that at low floods, all the weir bays shall be opened equally. Ever since the barrage is constructed, the maximum flood observed in the barrage is of the order of 4,000 cumecs and the designed flood never passed through the barrage. The low floods were consequently being passed by partial gate openings as per regulation proposed. This necessitated a few centimetres of opening of each gate. As the canal feeds the power house the pond level has to be maintained. As a consequence of these, the boulders were all trapped in the upstream and there is heavy silting and serious aggredation in the downstream also filling the cistern completely.

The silting has become so acute that, per chance, the maximum flood passes through the barrage, it may overtop the banks resulting in serious consequences.

Remedial measures are now being studied at the U.P. Irrigation Research Institute, field station at Bahadrabad, for a better functioning of the barrage. As a first measure it is decided to modify the regulation. It is now proposed to open a couple of weir bays completely instead of all the bays partially, so that the flood is discharged at the maximum intensity of the bay. This would make the boulders trapped in the pond, roll down the barrage.

A second proposal under study is to close some of the bays by a bund and divert the river through the required water-way. A breaching section shall be introduced in the bund in the vent of any designed flood occurring, could pass through the close bays.

5.2.3. . Table 5.1 gives the details of water-way provided in some of the existing weirs and barrages.

5.2.3. . The overall water-way for the barrage having been fixed it needs to be decided upon the water-way for the undersluices. It is a common practice to "think" of the water-way for the undersluices on the following considerations:

i) They should have sufficient capacity to flush the still pond above it. Capacity at least double the canal discharge is considered desirable.

ii) They should be able to pass winter freshets and low floods except during the rainy season without necessity of dropping the weir shutters.

iii) They should be able to handle 10% to 15% of the maximum flood discharge to reduce the length of the remaining portion of the weir or barrage.

The discharge intensity in the undersluices is thus kept higher than the rest of the structure. But it can be seen from table 5.1, that the above norms cannot be used as a rigid criterion. As much as 36.8% of the discharge is accounted by the undersluices in the Gandak Barrage, 37.9% on the Wazirabad Anicut on the Yamuna, and only 12.85% through the Trisuli.

The dimensions of the undersluices, the percentage of discharge to be passed through them, and the water-way as a whole for the barrage - are best determined by the help of models and hence it may not be possible strictly to lay down any definite rule for them. The preceding considerations at best can be used as guiding principle as a basis of detailed studies.

5.2.4. Crest level

As already stated, the crest level has a direct bearing on the water-way. In fact, the afflux, discharge intensity or water-way and the crest level are inter-related and a suitable combination has to be evolved keeping in view the limits for each.

In general, the crest levels are fixed on the consideration of the existing bed levels at the barrage site. The undersluice crest is kept usually at or near the bed level in the deepest channel, and is lower than the barrage crest to attract deep currents in front of the regulators. To arrive at this level it would be necessary to study the river cross-section of several years.

It is desirable to keep the crest levels as low as possible, so as to have minimum interference with regime of the river and to restrict its deepening tendency. A permanently-raised crest will result in a higher afflux and is also likely to result in loss of control on the river. Lower crest level results in lesser afflux during high floods, but this results in increase in cost of works, due to increase in effective head over floor resulting in increase in height of gates, increased thickness of floors and increased cost of super-structure above floor levels.

It is therefore necessary to make alternative studies with different upstream floor levels of undersluices and barrage bays. The downstream floors for these various crest levels have also to be determined from economic considerations.

Interesting studies have been made in regard to crest levels for the undersluices and weir bays in case of the Girija Barrage, of the "Sarda Sahayak Pariyojna."(12) The alternatives with different crest levels that were studied are -

i) Undersluice crest and upstream floor at EL 130.30,(m) other barrage bays, crest and upstream floor at EL 130.90;

ii) Undersluice crest and upstream floor at EL 129.50, other barrage bays, crest and upstream floor at EL 130.50;

iii) Undersluices,upstream floor at EL 128.50 with raised crest at EL 129.50. Other barrage bays, upstream floor at EL 129.50 with raised crest at EL 130.50;

iv) Undersluice crest and upstream floor at EL 128.50, other barrage bays, crest and upstream floor at EL 129.50.

Studies have revealed that there is very little change in the cistern levesl and afflux (having a range of 0.9 m to 0.75 m). The performance of the jump also did not change materially.

Alternative (iv) with raised crest was adopted for the following reaons:

(i) Due to crests being at EL 129.50 and 130.50 as in alternative (ii) the height of gates and hence cost has not increased;

(ii) Due to upstream floors being 1.0 m lower than crest the river bed can stabilise at lower levels as in alternative (iii).

Thus alternative (iv) combines the advantages of both alternative (ii) and (iii). Due to provision of raised crests the coefficient of discharge also improves.

5.2.5. Likely retrogression

The river regime is affected by the construction of a diversion work across its channel in the following ways:

i) The slope of the river upstream of the structure flattens due to the ponding up of waters;

ii) An increase in tortuosity, as a result of ponding up, as the bulk of the silt charge of the river water deposits in the pond, leading to the formation of irregular shoals;

iii) A progressive degradation or retrogression of bed levels downstream, due to the picking up of bed silt by the relatively silt-free water escaping over the structure.

The above effects develop and continue for a number of years. As shoal formation in the upstream reach increases, the resistance to flow of river is increased due to the tortuous route that the water has to take round-about the shoals. To overcome this resistance, increased head is required and the river starts to regain its original slope thus extending the effect of afflux further upstream.

A stage will be reached when the upstream pond will absorb no further silt-burden. As the off-taking canal takes comparatively silt-free water, the silt burden will go downstream while the discharge going down will be below normal. This will lead to silt deposit and a long-range recovery of levels on the downstream side.

This aspect of changes in the regime of a river caused by the construction of a diversion work must be considered in their design.

In the first few years after the construction of the weir, the bedlevels downstreamof the barrage would be lowered and it would be rapid and progressive. This is known as retrogression. Retrogression may undermine the stability of a work by an increase in the exit gradient beyond the safe limits. Generally, the low water-levels have been found to drop from 1.22 m to 2.15 m, while the maximum, only by 0.30 m to 0.45 m.

The following figures for retrogression have been assumed in some of the important barrage in India:

i) Narora Barrage

Discharge	Retrogression
Above 11,300 cumecs (4 lakh cusecs)	nil
Between 7080 cumecs and 11,300 cumecs	0.304 m
Below 7080 cumecs	0.61 m

ii) Kosi Barrage

Above 26,900 cumecs (9.5 lakh cusecs)	0.456 m
21,250 cumecs	0.58 m
14,450 cumecs	1.035 m
8,500 cumecs	1.37 m
2,830 cumecs	1.61 m

iii) Ramganga Barrage

0.91 m upto maximum discharge of 7,350 cumecs and 1.52 m for discharges 565 cumecs and below. Linear variation has been assumed for intermediate discharges.

iv) Dakpthar Barrage

Linear variation of 0.61 m at 14,450 cumecs, to 1.83 m at 706 cumecs.

v) Gandak Barrage

Linear variation from 0.472 m at 24,075 cumecs, to 1.22 m at 2,830 cumecs.

vi) Ashan Barrage

Linear variation from 0.6 m at 4,500 cumecs, to 1.20 m at 300 cumecs.

vii) Girija Barrage

Discharge (cumecs)	Retrogression (m)
26,500	0.30
22,200	0.45
19,700	0.60
10,000	0.90
6,000	1.20

viii) Salandi Barrage

Linear variation of 0.31 m at 3,200 cumecs, to 1.37 m at 283 cumecs.

ix) Nangal Barrage

Linear variation of 0.61 m at 10,500 cumecs, to 1.325 m at 1,415 cumecs.

The above figures indicate that no rule can be laid down for the retrogression to be assumed in design.

In case if a barrage is located below a reservoir, the retrogression figures may be slightly high for low discharges since most of the silt will be entrapped in the upstream reservoir.

5.2.6. Concentration of flow

It is customary in design of weirs to assume a certain percentage of discharge intensity over and above the normal intensity of discharge obtained by dividing the total discharge in a particular section of the weir by the length of crest in that section, to allow for any possible concentration of flow. This figure adopted for various barrages, varies. Some of the values as assumed for design purposes in the existing barrages, are as below:

1) Narora Barrage

a) For design of cistern level and sheet pile depth: 5% concentration over design flood of 14,150 cumecs, both for undersluices and barrage bays.

b) For protection works: no concentration of flow.

2) Kosi Barrage

a) For design of cistern levels and sheet pile depth: 20% concentration over design flood of 26,900 cumecs for undersluices and barrage bays.

b) For design of protection works: 20% concentration over design flood for barrage bays and no concentration for undersluice bays.

3) Ramganga Barrage

a) For design of cistern levels and depth of sheet piles: 10% concentration over design flood of 7,350 cumecs for undersluice bays and 20% concentration for other barrage bays.

b) No concentration of flow for protection works.

4) Dakpathar Barrage (across Yamuna)

a) For design of cistern levels and depth of cutoffs in undersluice and barrage bays:

i) For discharge upto 8,500 cumecs, 20% concentration;

ii) For discharge between 8,500 cumecs and 11,600 cumecs, 15% concentration;

iii) For discharge from 11,600 cumecs to 14,450 cumecs, 10% concentration.

b) No concentration of flow was assumed for the protection works.

5) Gandak Barrage

a) For design of cistern level and cutoffs: 20% concentration over design flood of 24,075 cumecs for undersluic bays and other barrage bays;

b) No concentrationof flow for the protection works is assumed.

6) Ghagra Barrage

a) For the design of cistern levels and sheet piles for both undersluice and barrage bays: 20% concentration over design flood of 22,200 cumecs and all other lower floods. No concentration was allowed in checking for super-flood of 26,500 cumecs.

b) No concentration of flow was allowed for protection works.

7) Nangal Barrage

a) Forthe design of cistern:

i) 20% concentration upto 5,675 cumecs

ii) 10% concentration for discharges 5,675 cumecs to 7,075 cumecs

iii) 5% concentration for discharges 7,075 cumecs and above.

b) No concentration was assumed for protection works.

5.6. Regulation

5.6.1. For proper design of pockets it is necessary to decide the method of Regulation. The width of the pocket should be such that velocity in river/velocity in pocket (V_R/V_P) ratio would be greater than unity at the critical discharge when appreciable bed movement takes place in the river.

The purpose of creating pockets is to ensure the entry of clear water, into the canal heads. Two methods of operating canals taking-off upstream of a barrage are in practice.

i) In the first method known as the ''still pond regulation'' all the gates in the pocket are closed and the discharge drawn into the pocket is limited to that of the canal requirements, the surplus being escaped over some other section of the weir. The velocity of water in the pocket is, therefore, very much reduced, as a smaller discharge enters through the same water-way. The silt is then enabled to settle down and relatively clear water enters the canal. This system is practicable when the canal head regulator has a high crest above the upstream floor of the undersluices. Accumulation of silt results and this is allowed upto a level, 0.5 m below the crest. The silt is then washed off by opening the sluices, the canal being closed.

ii) In the second method known as the ''semi-open flow,'' excess discharge is allowed into the river by keeping pocket gates partly open, the top water passing into the canal. The two streams of water have to divide out in front of the head regulator, one passing into the canal and the other escaping down the undersluices.

The relative merits of the two methods have been discussed by D. V. Joglekar(15) which is as below:

With 'still pond operation', the pocket gates are not useful for passing low flood discharges, while with 'simi-open flow', a part of the discharge can be directed through the pocket. So far as exclusion of sand is considered, 'still pond operation' has a decided advantage over the other method. On the other hand, it was advocated by the supporters of 'semi-open flow' method that the forward velocity created by partly opening the gates in the pocket carried the coarse bed material into the river and relatively clearer water flowed into the canal. This has been proved to be wrong.

Secondly, to operate 'semi-open flow' some more discharge than required for the canal has to be passed into the pocket. This adversely affects V_R/V_P ratio and also draws relatively more sand in the pocket. Thus semi-open flow especially at Low Stages of river discharge does more harm than good. This is, however, possible without detrimental effect on exclusion when the river discharge is high and some more discharge into the pocket would not materially affect V_R/V_P ratio. This also helps to reduce pond level during floods which is undoubtedly advantageous in reducing the afflux and therefore ponding-up effect.

If the river discharge is such that V_R/V_P ratio is unfavourable, then this can be improved slightly by opening the gates adjacent to the divide wall, more than the gates further away from it.

5.6.2. ./

5.6.2. Regulation at diversion works where canals take-off from both sides

At a number of diversion works canals take off from both sides, such as -

1) Sukkur Barrage
2) Madhopur Barrage
3) Rupar Barrage
4) Ferozpur Barrage
5) Kotri Barrage

For proper regulation, where canals take off at both ends, undersluices with depressed crest are provided at either end. Both the undersluices have separate divide walls to form respective pockets. With the help of depressed undersluices at either ends, attempt is made to maintain a deep channel towards either side. Still-pond system is maintained in both the pockets and regulation is done with the help of weir bays. Although identical conditions are maintained at both sides, one or the other canal draws greater proportion of sediment. The cause for this unsymmetrical distribution is the river approach. It is seldom similar and rarely straight. The canal situated on the concave side draws less silt and that on the convex side draws more silt.

In the note of 'Sediment exclusion - Nature's Way' by Central Water & Power Research Station, Poona, it is stated that if canals take off from both sides of a barrage and if V_R/V_P ratios are unfavourable, they can be improved by keeping opening of gates of main spillway minimum in the centre, and increased gradually towards the divide walls. This is known as the double wedge system of regulation. If semi-open flow is adopted, then gates away from the head-regulator should be opened more than the nearer ones.

5.6.3. However, each structure will have its own method of regulation, which would be suitable from all points of view. In this case the regulation adopted at the Sarda Barrage and the Sone Barrage are interesting.

5.6.3.1. Regulation at the Sarda Barrage

The Sarda Barrage consists of 34 bays divided into six compartments. First compartment consists of the four under-sluices - bay nos. 1-4 - and the other five compartments consist of six barrage bays each. When the supplies to be escaped down the barrage are 566.4 cumecs or less, the same, as far as practicable be passed through the undersluices with gate Nos. 1 to 4 so regulated that at least 283 cumecs, or the entire supplies if less than this fugure are passed equally through gate Nos. 1 & 2 and the balance equally through gate Nos. 3 & 4.

Supplies in excessof 566.4 cumecs are distributed evenly in all the bays from 5 to 34 as far as possible upto a maximum discharge of 3,398 cumecs and beyond that, all the six ompartments should draw equal water as far as possible.

The above instructions are subject to certain limitations, viz. -

1) The difference in head on the two sides of upstream groynes and piers should not exceed (1.5 m);

2) In each compartment, the gates must be opened in any order best suited to minimise the action on the groynes or long piers and to prevent parallel action along the upstream face of

the barrage.

5.6.3.2. Sone Barrage

For the purpose of regulation the barrage has been divided into 2 units as shown below:(16)

Undersluices

Left	-	8 vents	Unit 1
Right	-	4 vents	

Spillway

57 vents	Unit 2

The following points were considered for the operation of gate for various discharges:

1. Undersluice gates are kept closed as much as possible. Toprevent over-topping of gates, these are, however, opened, when the water level on upstream side exceeds pond level, which occurs when the river discharge exceeds 11,330 cumecs. The height of the opening is thus kept - upstream W.L. - Pond Level, to have the same free board for the gates as obtained at pond level condition. When the river discharge exceeds 34,000 cumecs the gates are,however, fully opened, as the spillway is incapable of passing this discharge without causing an afflux of 1.22 m. The canal is closed when river discharge exceeds 34,000 cumecs.

2. The velocity in the pockets should be less than Lacey's critical velocity ($V = \sqrt{2/5} . \sqrt{fR}$)

3. The velocity in the pockets V_p should be less than velocity in the river.

These limitations have a clear implication on the design of undersluices etc. (i.e. on the cistern and the waterway) as can be seen from the regulation of the two barrages.

5.7. Canal closure

5.7.1. Closure of the canals is one of the measures for sediment exclusion at diversion works. In certain river stages, when the stream is very heavily charged with sediment, the canal is closed at the head in order to prevent the material entering into the canal. The canal is also closed when the level of the bed in the pocket is so high that the material is picked up from there and goes into canal. As soon as the sediment entry to the canal increases a closure is effected. The closure of the canal pertaining to sediment control may be, flood and sluicing closures etc.

5.7.2. Flood closure

In floods when the sediment charge in the river is very high, the canals are closed at headworks to stop the sediment-laden water going into the canals. The river discharge at which the sediment entry gets very excessive is different at different headworks and mostly depends on the nature of catchment, location and design of headworks and approach conditions.

5.7.3. Sluicing closures

Sluicing closures are effected to flush out the pocket. The sluicing closures may be normal sluicing closures and special sluicing closures.

5.7.3.1. Normal sluicing closure:

In May or June, when the river discharge begins to rise, the canal is closed for a short period and the undersluice gates raised to clear the approach channel and flush the pocket. Similarly, another closure is effected at the end of the monsoon, i.e. in September, so that the approach is cleared off for feeding the canal during winter.

5.7.3.2. Special sluicing closures:

The sand or gravel forms a ramp in front of the head-regulator. In order to lower the height of the ramp below the H.R. crest, the canal at certain headworks (like Madhopur) is closed for a short period, say from 15 minutes to a couple of hours, and the undersluices opened to wash the ramp. This reduces the quantity of sediment entering the canal.

5.7.4. Some of the important conclusions drawn in regard to closure are as below:

1) Practical experience shows that a sluicing closure for 15 to 30 minutes is more useful for flushing the pocket than a longer closure of two hours or more.

2) The sluicing closure effected in low river discharge is not very effective. A discharge between 283 to 566 cumecs is more suitable.

3) The sluicing closure done in very high river stages as floods is also not very useful. Even if the ramp is washed away, it re-forms immediately after the canal is re-opened.

4) The length of the pocket affected by a sluicing closure would depend upon intensity of discharge, depth, velocity, size distribution of bed material.

5) Partial closure by reducing the canal discharge, at the same time opening the undersluices, is not as useful as full closure.

5.8. Specific problems

The water-way for the Mundali Weir across Mahanadi is fixed from the considerations discussed in the following paragraphs. It is interesting to note that the bed load transportation of the river governed the criteria for fixation of the water-way.[40]

The Mahanadi in Orissa, bifurcates into two main branches i) the Main Mahanadi, ii) the Katjuri. Just upstream of the bifurcation, at Naraj, a weir was constructed for distribution of the flood discharges into the two branches. (Fig. 5.2)

During the 2nd Five-Year Plan for the development of irrigation, a weir was proposed about 5 km upstream of the Naraj Weir.

While fixing the water-way for the weir one of the criteria in view was that the construction of weir should not, as far as possible, alter the distribution of bed-load amongst the two branches, as obtained under the existing conditions. This was necessary in view of the existing downstream canal system, taking off from old weirs on the branches. Under the existing conditions, the Katjuri branch was seen to draw 45.4% of the total bed load.

The length of the weir as proposed by the Designs Organis-

-ation was (C.W. & P.C.) 1061 m (overall) for a maximum flood discharge of 34,200 cumecs. Tests with 19,280 cumecs discharge, all the bed load from the undivided river was drawn into the Katjuri. Tests with other alignments tilted at 10°, 12° on the upstream and 8° on the downstream to the original alignment did not improve the distribution of the bed load. Further tests made with 1220, 1370, 1520-m lengths of the weir, were not satisfactory, insofar as the bed load drawn by the Katjuri was much higher than that under existing conditions. Even with 1520 metres long weir, the bed load into the Katjuri was 54.5%, under 6.8 lakh cusecs flood stage, as against 45.4% under existing conditions. Tilted alignments with longer lengths were also not satisfactory. A trial-and-error ultimately fixed the waterway at 1,335 m for a maximum discharge of 34,200 cumecs and looseness factor of 1.34.

Discharge intensities:

The weir is located across the river just below a bend with the main channel along the right bank. The discharge intensities along the weir will not, therefore, be uniform. An experiment was conducted for finding the actual intensities of discharge in each 132.5-m segment of the weir. The table below gives the discharge intensities for a flood stage of 34,200 cumecs. It can be seen that the intensities of discharge varied in the various segments of the weir. The intensity reduced considerably at the left flank, thus providing justification for the necessity of accounting for concentration of flow in design. Further considerable economy could be achieved if in the design

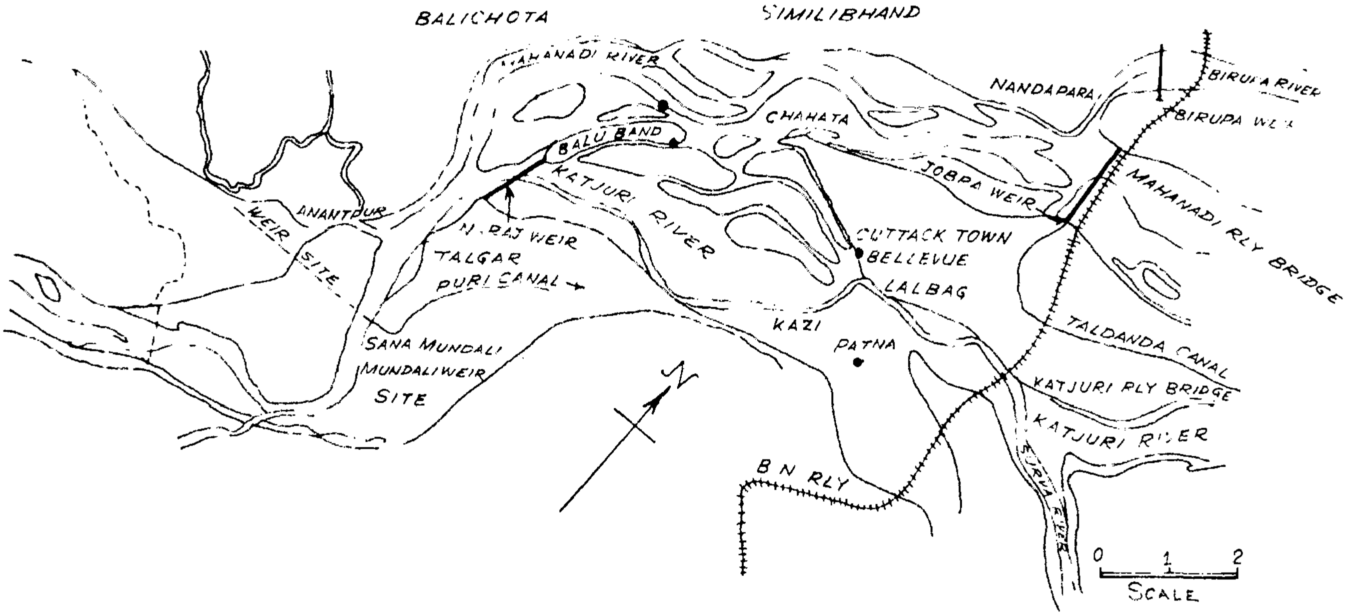


FIG. 5.2 INDEX PLAN OF MUNDALI WEIR.

of loose aprons, actual intensities were considered.

Item	Discharge intensity cumecs/m	% Discharge distribution
Pocket sluices	32.3	8.44
Undersluices	38.2	13.26
Main weir portion:		
0 - 152.5 m	34.4	15.08
152.5 - 305 m	34.5	15.1
305.0 - 457.5 m	25.1	11.18
457.5 - 610 m	24.4	10.7
610.0 - 762.5 m	21.4	9.44
762.5 - 915 m	19.2	8.45
915.0 - to left bank	17.3	7.6

Chapter 6

HYDRAULIC DESIGN OF BARRAGES AND WEIRS

6.1. Technique of Barrage or Weir design was greatly benefited from the study of various such structures which have failed.

The design can be broadly classified into two parts:

i) Hydraulic design

ii) Structural design

The structural design consists of dimensioning of the various parts of the structure to enable it to resist safely all the forces acting on it. This is dealt with in Chapter 7.

The hydraulic design comprises evaluation of the hydraulic forces on the barrage and determination of the profile of the barrage consistent with economy and functional efficiency.

6.2. The hydraulic design is governed by the following two conditions:

i) Surface flow

ii) Sub-surface flow

6.3. By surface flow we determine the profile of the barrage, viz. -

a) The level and length of the downstream floor and the glacis;

b) The level and length of the upstream floor;

c) Crest level and water-way (these have already been discussed);

d) Likely scour depth and depth of sheet pile;

e) Length and thickness of loose aprons.

6.3.1. Downstream floor

6.3.1.1. From a practical view point the hydraulic jump is useful means of dissipating excess energy in super-critical flow. Its merit is in preventing possible
for it can quickly reduce the velocity of the flow beyond the paved apron through dissipation of energy in the jump, to a point where the flow becomes incapable of scouring the downstream channel bed.

The formation of hydraulic jump used for energy dissipation, is usually confined, as far as possible, entirely in the channel reach, below the glacis, that is known as the stilling basin. The bottom of the basin is paved to resist scouring. In practice the floor of stilling basin is seldom kept deep enough for formation of the free hydraulic jump under all conditions on the paved apron, because such a basin is generally too expensive. Consequently, accessories to control the jump with raised basin, are usually installed in the basin.

6.3.1.2. Types of hydraulic jumps:

A hydraulic jump may form when water moving at a super-critical velocity in a comparatively shallow stream strikes water having a substantial depth and subcritical velocity.

There are essentially five different forms of the hydraulic jump which may occur on a horizontal apron and which may be encountered in the designof energy-dissipation devices. These are classified according to the Froude number 'F',(17) which is the ratio between the inertial forces and gravity forces given by

$$F = \frac{V}{\sqrt{gy}}$$

where V = velocity of flow

y = hydraulic depth

For $F_1 = 1$, the flow is critical, and hence no jump can form.

For $F_1 = 1$ to 1.7, the water surface shows undulations, and the jump is called an undular jump. (fig 6.1)

For $F_1 = 1.7$ to 2.5, a series of small rollers develop on the surface of the jump, but the downstream water surface remains smooth. The velocity throughout is fairly uniform, and energy loss is low. This jump may be called a weak jump.

For $F_1 = 2.5$ to 4.5, there is an oscillating jet entering the jump bottom to surface and back again with no periodicity. Each oscillation produces a large wave of irregular period which, very commonly in canals, can travel for miles doing unlimited damage to earth banks, and ripraps. This jump may be called an oscillating jump.

For $F_1 = 4.5$ to 9.0, well-stabilised jumps are formed. If possible, structures should be designed to ensure that a jump in this category will be formed. In this range, the energy dissipated by the hydraulic jump will vary from 40 to 70%.

For $F_1 = 9.0$ and larger, the high velocity jet continues downstream for a long distance, with a considerable amount of spray and rough water resulting. The energy dissipation may reach in this case, to 85%.

The limits of the Froude's number indicatedfor various forms of the jump are not definite and may overlap to a certain extent depending on local conditions.

6.3.1.3. Height and length of the hydraulic jump:

The height of the jump can be defined as the difference between the depths of water upstream and downstream of the jump, denoted by h_j.

Thus $h_j = d_2 - d_1$

It can be seen that the length of the hydraulic jump (L_j) bears a definite relationship to its height.[18]

The length of the hydraulic jump is of particular significance since it is the principal factor in determining the length of stilling basins. The longitudinal element of the jump is, without doubt, the most difficult element to measure. This is because of differences in opinion as to exactly where the terminus of the jump lies. In addition, there is a leck of agreement among investigators as to the definition of the length of the jump.

Thelengths of the jump as found by different investigators are as follows:

i) Bakhmeteff & Matzke (1932-33)	$L_j = 5\ (d_2 - d_1)$
ii) Smetana (1935)	$L_j =$ approx. $6(d_2 - d_1)$
iii) Kinney (1935)	$L_j = 6.02(d_2 - d_1)$
iv) Douma	$L_j = 3\ d_2$
v) Wu (1944)	$L_j = 10\ (d_2 - d_1) F_1^{-0.16}$
vi) Page (1935)	$L_j = 5.6\ d_2$

In 1954, a series of measurements to determine the length of the hydraulic jump was made by the U.S.B.R. It was the intention of the U.S.B.R. engineers to judge the length of the jump from a practical stand-point which would best represent the end of the concrete floors and side walls of a conventional stilling basin. In the experiments the Froude number was varied from 2 to 20.

An analysis of the experimental data indicates that a good relationship exists between the length and height of the jump and this was established as 6.9 times the jump-height.

6.3.1.4. Basin apputenances:

Appurtenances such as chute blocks, baffle piers, and end-sills are often installed to help increase the performance of a stilling basin. In addition, they are helpful in stabilis- the flow, increasing the turbulence and distributing the velocities more evenly throughout the basin. In some cases, a reduction in the required tail-water depth and length of the basin may be possible by the addition of appurtenances.

The appurtenances used are -

a) Chute blocks

b) Baffle piers

and c) End sills

a) Chute blocks are installed at the entrance of the stilling basin to increase the effective depth of the entering stream, to break up the flow into a number of jets and to help create the turbulence required for energy dissipation. The blocks also tend to lift the jet off the floor and result in a shorter basin-length, than would be possible without them.

b) Baffle piers are installed in the stilling basin principally to stabilise the formation of the jump and increase the turbulence, thereby assisting in the dissipation of energy. The employment of baffles will be helpful in reducing the tail-water depth required and also in shortening the basin-length.

c) End-sill is a vertical, stepped, sloped, or dentated wall constructed at the downstream endof the stilling basin. The purpose of the end-sill is to lift the flow off the river bed and create a back current, which causes bed material to be transported and heaped up against the back face of the sill. Laboratory tests indicated that the sill greatly increases the efficiency of the basin.

6.3.1.5. Design procedure:

The following relationships between the various parameters of the jump have been established:

$$H_L = \frac{(d_2-d_1)^3}{4d_2.d_1}$$

where H_L is the head lost through energy dissipation provided by the hydraulic jump and

$$d_2 = \frac{d_1}{2} = \sqrt{\frac{2q^2}{d_1 g} + \frac{d_1^2}{4}}$$

and $$d_c = \left| \frac{q^2}{g} \right|^{1/3}$$

where q is theintensity of discharge.

Blench's curves were used in the design of most of the old weirs and barrages for determining the stilling basin level. Blench has prepared curves relating H_L and E_{f2} (the downstream specific energy) for various intensities of discharge. (Plate XI-2; CB.I.P. No. 12) For a particular intensity of discharge, the head loss H_L is determined, which is the difference between the upstream and downstream T.E.L. (after accounting for retrogression). The downstream specific energy E_{f2} is then read forthe corresponding H_L from the Blench's curves. The upstream specific energy E_{f1} is then given by $E_{f1} = E_{f2} + H_L$. The level at which thejump would form is then obtained by subtracting E_{f2} from the downstream total energy line. The pre-jump depth, d_1 , and the post-jump d_2 are then read from Energy of flow curves; which are prepared from the basic relation:

$$E_{f1} = d_1 + \frac{q^2}{2g\ d_1^2}$$

$$E_{f2} = d_2 + \frac{q^2}{2g\ d_2^2}$$

The Central Water & Power Commission adopts the following procedure for determining the downstream floor level:

For a particular intensity of discharge the upstream T.E.L. is determined. Ann appropriate level is assumed for the stilling basin.

Now E_{f1} = U/S T.E.L. - R.L. of the basin

$$= d_1 + \frac{q^2}{2g\ d_1^2}$$

d_1 can then be determined.

F_1 isthen given by

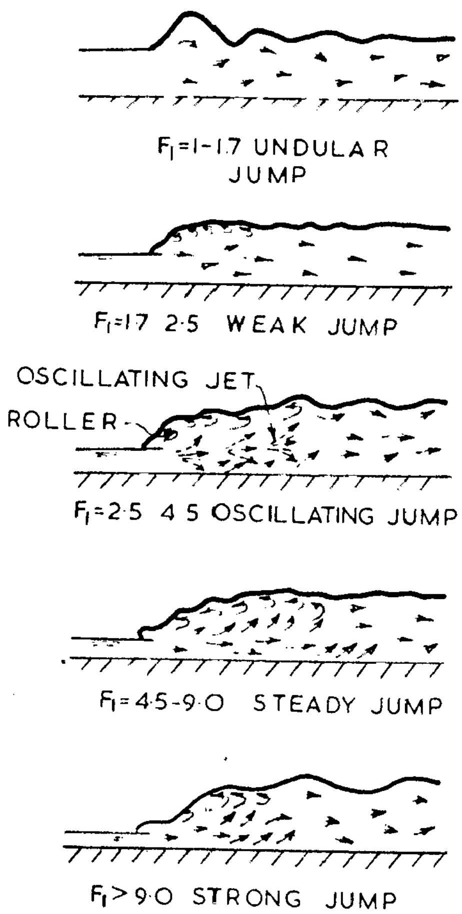


FIG. 1.9

VARIOUS TYPES OF HYDRAULIC JUMP .

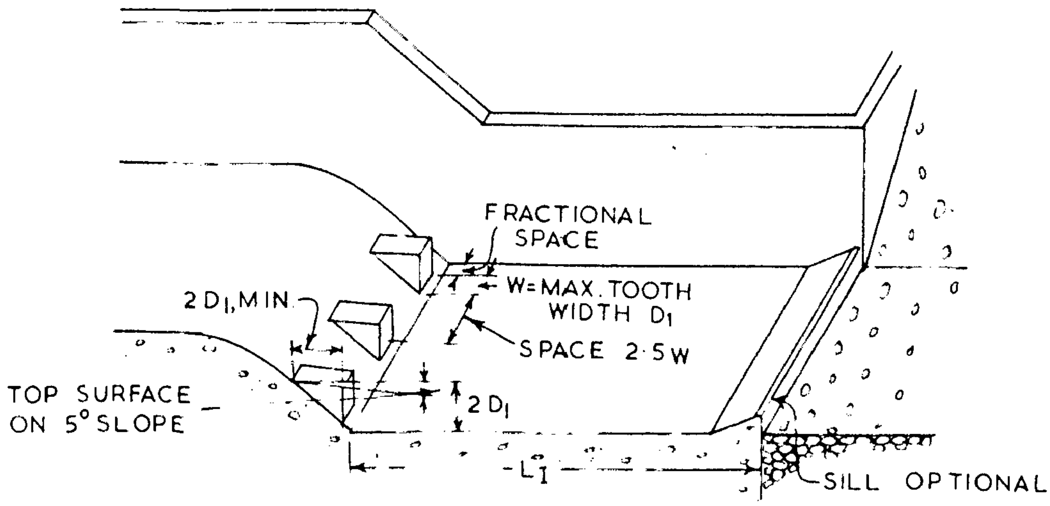
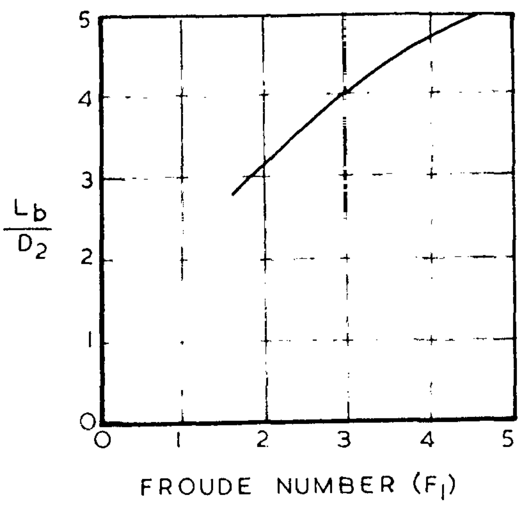
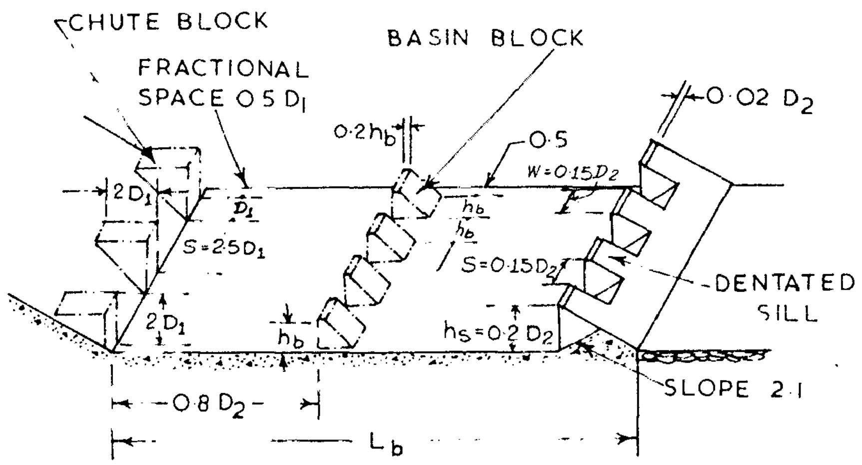


FIG. 6.3

PROPORTIONS OF USBR BASIN IV. (U.S BUREAU OF RECLAMATION [34])



A. RECOMMENDED LENGTH FOR BASIN I



B. APPURTENANCES FOR BASIN I

FIG. 6.2 DIMENSION SKETCH FOR BASIN I

$$F_1^2 = \frac{q^2}{g \; d_1^3}$$

d_2 , the conjugate depth is then calculated by the formula

$$d_2 = \frac{d_1}{2} \quad (-1 + \sqrt{1 + 4.44 \; F_1^2}$$

The stilling basin level is then given by (d/s water level - d_2).

The level so calculated should be equal to the assumed level orelse the procedure is repeated.

In both the methods no attempt is made to raise the floor level to take advantage of ancillary works.

The I.S. Code[19] follows the same above procedure or makes (fig 6·2) use of curves. It recommends type I basin for barrages. According to the Code the basin floor, generally, should not be raised above the level required from segment-depth consideration. If the raising of the floor becomes pbligatory due to site conditions, the same should not exceed 15% of d_2 and the basin in that case should be further supplemented by chute blocks and basin blocks.

Eleveutorski recommends that with the addition of appurtenances in the stilling basin the hydraulic jump can be formed with a tail-water depth equal to $0.9y_2$, thereby permitting reduction in the required tail-water depth.

The U.S.B.R.[20] recommends type IV basin which chute blocks and end-sill. The tail-water depth can be kept upto 10% greater than the computed conjugate depth.(fig 6·3)

6.3.1.6. ../

6.3.1.6. The lengths of stilling basin as proposed by different investigators have been indicated in para 6.3.13.

Theolder practice adopted in most of the barrages and weirs is to fix the theoretical length of the stilling basin as $5(d_2-d_1)$.[8] This corresponds to the recommendation of Bakhmeteff and Matzke. No effect, probably, has been taken of the appurtenant works-

Elevatorski[18] recommends a length of basin equal to $6.9(d_2-d_1)$without any appurtenant works and with appurtenances the length can be reduced to $4.5(d_2-d_1)$.

Data on horizontal stilling basins below existing weirs and barrages is given at table 6.1. It can be seen that the ratio of the length of the basin to the height of the jump in most cases varies for the same type of appurtenances. Even for the undersluices and the other bays the ratios are not same. This shows the dimensions of the appurtenances have a great influence.

The I.S. Code does not correlate the length of the basin with the jump height. Rather it provides a curve correlating the Froude's number with L_b/d_2. The maximum ratio of L_b/d_2 is 5.0 and the minimum 2.7, for Froude Numbers 4.5 and 1.7, respectively. For the Sarda and Kosi Barrages, which have the same Froude's number, the L_b/d_2 ratios are 2.75 and 3.42, respectively. But as per I.S. Code, the value from the graph is about 3.25.

Also in case of Narora undersluices, the Froude number is 2.02 and L_b/d_2 is 4.22. From I.S. Code curve, this value is 3.18.

Table 6.1 - Da

Sl. No.	Name of	$\frac{L_b}{D_2}$	$\frac{L_b}{D_2-D_1}$	Details of basin appurte-nances provided: Chute blocks	Basin blocks	Dentated sill
1.	Ashan Ba	3.68	4.74	Yes	No	Yes
2.	Yamuna:	3.71	5.26	No	No	Yes
		3.60	4.98	No	No	Yes
3.	Narora:	4.22	7.18	Yes	Yes	Sill blocks provided
		3.58	5.68	Yes	Yes	Yes
4.	Gandak:	3.84	12.5	No	No	Yes
		4.18	10.9	No	No	Yes
5.	Sarda:	3.17	4.95	No	No	Yes
		2.75	4.22	No	No	Yes
6.	Phika:	2.67	3.59	Yes	Yes	No
		2.70	3.36	Yes	Yes	No
7.	Kosi:	3.12	4.60	No	Yes	Yes
		3.42	5.38	No	Yes	Yes

We may, therefore, infer from these, that the empirical calculations are at best a preliminary guide and the actual dimensions of the basin have to be determined from model experiments, to suit particular local conditions. The experiments conducted at the Central Water & Power Research Station, Poona, (21) for the Hasdeo Barrage, across river Hasdeo in Madhya Pradesh, provide ample justification in this regard. It can be seen that a raised end-sill was only effective than the friction and chute blocks.

6.3.1.7. Hasdeo Barrage is constructed across the river Hasdeo, one of the main tributaries of the river Mahanadi in Madhya Pradesh. The details of the barrage are -

i)	Length between abutments	= 283.76 m
ii)	No. of spans	= 14 spans of 18.28 m each
iii)	Design flood	= 14,158.4 m^3/sec.
iv)	Super flood	= 19,821.8 m^3/sec.
v)	H.F.L. = R.L.	= 932.00
vi)	Crest Level of the spill-way	=R.L. 915.00
vii)	D/S Pavement Level	= R.L. 900.00
viii)	Length of the Pavement	= 14.03 m
ix)	Height of end-sill	= 1.52 m

Experiments were conducted for all the river discharges ranging from 2,831.69 m^3/sec. to 19,821.8 m^3/sec. including 20% concentration in steps of 2,831.69 m^3/sec. At super flood of 19,821.8 m^3/sec., it was observed that a standing wave formed almost at the end of the pavement and on the end-sill. This was an

Table 6.1 - Data on Horizontal Stilling Basins below existing weirs and barrages

Sl. No.	Name of structure		Design Discharge (cumecs)	Discharge per metre length (cumecs)	f (Silt factor)	D_1	D_2	F_1	L_b	$\frac{L_b}{D_2}$	$\frac{L_b}{D_2-D_1}$	Details of basin appurtenances provided: Chute blocks	Basin blocks	Dentated sill
1.	Ashan Barrage		4,500	21.1	8	1.55	6.91	3.50	25.39	3.68	4.74	Yes	No	Yes
2.	Yamuna:	Undersluices	14,440	40.4	4	2.80	9.45	2.75	35.05	3.71	5.26	No	No	Yes
		Other bays	14,440	31.4	4	2.83	8.47	2.80	30.48	3.60	4.98	No	No	Yes
3.	Narora:	Undersluices	14,160	27.9	0.846	2.68	6.50	2.02	27.48	4.22	7.18	Yes	Yes	Sill blocks provided
		Other bays	14,160	18.2	0.846	1.89	5.12	2.25	18.32	3.58	5.68	Yes	Yes	Yes
4.	Gandak:	Undersluices	24,070	50.4	3	5.27	7.62	1.33	29.30	3.84	12.5	No	No	Yes
		Barrage	24,070	48.3	3	4.18	6.77	1.46	28.27	4.18	10.9	No	No	Yes
5.	Sarda:	Undersluices	16,990	46.5	3	3.44	9.60	2.33	30.48	3.17	4.95	No	No	Yes
		Other bays	16,990	39.0	3	3.11	8.87	2.27	24.38	2.75	4.22	No	No	Yes
6.	Phika:	Undersluice	1,048	13.0	2	1.22	4.79	3.38	12.80	2.67	3.59	Yes	Yes	No
		Barrage	1,048	10.9	2	0.96	4.40	3.36	11.89	2.70	3.36	Yes	Yes	No
7.	Kosi:	Undersluice	26,850	36.9	1.3	2.8	8.75	2.53	27.3	3.12	4.60	No	Yes	Yes
		Barrage	26,850	33.3	1.3	2.93	8.00	2.26	27.3	3.42	5.38	No	Yes	Yes

We may, therefore, infer from these, that the empirical calculations are at best a preliminary guide and the actual dimensions of the basin have to be determined from model experiments, to suit particular local conditions. The experiments conducted at the Central Water & Power Research Station, Poona,(21) for the Hasdeo Barrage, across river Hasdeo in Madhya Pradesh, provide ample justification in this regard. It can be seen that a raised end-sill was only effective than the friction and chute blocks.

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vii)	D/S Pavement Level	= R.L. 900.00
viii)	Length of the Pavement	= 14.03 m
ix)	Height of end-sill	= 1.52 m

Experiments were conducted for all the river discharges ranging from 2,831.69 m^3/sec. to 19,821.8 m^3/sec. including 20% concentration in steps of 2,831.69 m^3/sec. At super flood of 19,821.8 m^3/sec., it was observed that a standing wave formed almost at the end of the pavement and on the end-sill. This was an undesirable condition as with retrogression of the downstream bed. The standing wave would further move downstream, thereby causing damage to the structure. In orderto improve the flow conditions and to push the standing wave towards the toe of the glacis, friction blocks of standard sizes alongwith toe blocks were tried. There was practically no improvement. The designed pavement length of 14.03 m represents only 1.5 (d_2-d_1) approximately corresponding to the super-flood discharge of 19,821.8 m^3/sec. and was therefore found to be quite insufficient. It was therefore thought to increase the pavement length.

A pavement length of 21.33 m, i.e. about 2.5 (d_2-d_1) was tried first. There was very little improvement. The length was further increased to 30.48 m, i.e. about 3.5 (d_2-d_1). Under this condition, it was seen that the standing wave formed almost at the toe of the glacis under a discharge equivalent to 19,821.8 m^3/sec. Even though the wave was well contained on the pavement, it was likely to move downstream with retrogression of the downstream bed. It was, therefore, decided that the structure should be tested for a downstream retrogressed bed level 1.52 m lower than the deepest river bed, i.e. for alevel of R.L. 895.00 (ft.). Under this set-up, the standing wave jumped clearly out of the pavement under a discharge equivalent to super flood of 19,821.8 m^3/sec., indicating dangerous conditions to the structure. It was felt that position of the wave under such a condition could only be controlled by suitable level of the pavement. The pavement was lowered to R.L. 890 (ft.) by a series of quick tests performed with lower pavement levels.

Under this set-up, it was seen that conditions had much improved and safe upto a discharge equivalent to 14,158.6

m^3/sec. For discharges higher than this figure, conditions were not satisfactory. At super flood, the standing wave was formed below the pavement. Further tests were performed with friction and toe blocks to improve the flow conditions. There was no improvement. It was therefore decided to increase the height of the end-sill.

Flow conditions obtained under thisset-up were safe for all discharges except for a discharge intensity equivalent to 19,821.8 m^3/sec. + 20% concentration. The wave is well contained in the pavement for 19,821.8 m^3/sec., but with 20% concentration the wave travelled down and formed near the end of pavement, with a more pronounced secondary wave downstream. Normally, such a condition would not be acceptable. Some calculated risk may, however, be taken in view of the fact that such a flood may occur very rarely and forshort duration. In view of this, the lengths of 30.48 m with downstream pavement level at R.L. 890.00 and end-sill of 2.13 was recommended.

6.3.1.8. In the case of Salandi Barrage, a cistern level of 105.00 (ft.) was recommended from design point of view. The basin was provided with friction blocks and an end-sill at R.L. 111.00 (ft.). The floor was stiffened with dummy piers. Model experiments showed that the floor level could be safely kept at R.L. 106.00 and the friction blocks served no purpose. The height of end-sill was recommended at R.L. 109.00. This shows that the energy dissipation was achieved by the dummy piers.

Thus in the former basin, there is reduction in the length of the basin from the conventional limit by provision of appurtenant works and latter by an increase in thelevel.

6.3.1.9. Once the level and the length of the downstream floors are determined, the upstream length can be fixed, the floor length having been fixed from exit gradient considerations. The top width of the crest is fixed from practical considerations and is of the order of 1.85 to 2.5 m. The upstream slope to the crest is kept from 1:1 to 1:3.

6.3.1.10. For the glacis, slopes between 1:3 and 1:5 are considered to be most suitable both for the maximum dissipation of energy and economy.[8]

According to Elevatorski[18] when the slope is too steep, the high velocity of the water shoots out under the surface of the pool, making likely dangerous erosion. Relatively few experiments have been conducted in channels having steep slopes. Most designers have limited the slopes of stilling basin aprons, to a maximum slope of 4 horizontal to 1 vertical. But for a better coefficient of discharge a steep slope is recommended.

6.3.2. Upstream floor

Subject to the minimum depth of downstream vertical cut-off required from scour considerations, the depth of the cut-off and the total floor length of the barrage or weir can be mutually adjusted to provide the most economical and suitable combination to keep a safe exit gradient. The excesslength of floor over the requirements of the downstream floor,the glacis and the crest from hydraulic considerations, will form the upstream floor.

The level of the upstream floor is generally kept at the river bed level or slightly higher.

6.3.3. Likely scour depths and depth of sheet piles

Scour is an evil common to a variety of irrigation works, foremost among which are dams, barrages and weirs, built on large alluvial rivers. For design of structures, it was therefore only natural that the depths and slopes of the scour holes, recorded in major weirs and barrages have been studied for obtaining criteerion of design of structures. The scour hole should be prevented as a means of protecting the structure from caving and consequently endangering its stability. Scour is governed by surface flow.

The weir or barrage is generally protected against scour by vertical cutoffs at either ends of the pucca floor, which may be sheet piles or concrete walls depending on the sub-soil conditions. Adjoining the cutoffs, block and loose stone protections are provided.

Hence scour is guarded against by i) sheet pile cutoffs, ii) Block and loose stone protection.

The criteria laid down for the depth of sheet piles are as below:

i) Downstream sheet pile:

a) That with a suitable length of floor, it gives a safe exit gradient for a maximum head (This is discussed in section 6.4.2.

b) That its bottom should be nearly at or below the level of high flood scour, for that section of the work for which the depth of piles is being determined.

ii) Upstream sheet pile: This is determined only from scour considerations.

The depth of scour can be calculated from:

i) Kennedy's formula -

$$d = 1.82\ q^{0.61}$$

where,

d = depth of scour in metres

q = intensity of discharge in cumecs.

ii) Lacey's formula -

$$R = 1.35 \left| \frac{q^2}{f} \right|^{1/3}$$

where,

R = the scour depth in metres

f = Lacey's silt factor

q = intensity of discharge in cumecs

Lacey's formula[8] has been accepted in preference to that of Kennedy as the former is hydrodynamically more rational and takes note of different grades of bed material. Further, the formula gives somewhat higher values than Kennedy's and is considered on the safe side.

The following classes of scour are given by Lacey:

Class	Reach	Depth of scour
A	Straight	1.25 R
B	Moderate bend	1.50 R
C	Severe bend	1.75 R
D	Right-angled bend	2.00 R

Class A is likely to occur only where just below the loose aprons.

Class B is likely to occur anywhere along the aprons of guide banks in the straight reach.

Class C & D is likely to occur at and below the noses of guide banks.

For the design of sheet piles on upstream and downstream of barrage, the publication[8] recommends that it will be enough to take the piles to a depthof scour equal to R.

Leliavsky[5] states that a statistical examination of a number of works built on alluvial rivers shows that the depth of the downstream pile tends to approach, under certain assumptions, the natural downstream water depth in the river. It would therefore appear that the two (depth of downstream piling and water depth above floor level) must be more or less equal.

Table 6.2(A) shows the statement of depth of sheet-piles criteria in different barrages. It can be seen that for most of the barrages the criteria for scour depth in the upstream is taken as R and downstream, 1.25R, though the provisions have been, in certain cases, made on the higher side for the reasons stated in the remarks column. This higher provision also takes into account the extra provision made in the intensity of discharge for possible concentration of flow.

The parameters on which the scour depth determined by the Lacey's formula depends are -

i) The intensity of discharge q

ii) Lacey's silt factor f

It is obvious that higher the intensity, greater will be the

Table 6.2(A) - Statement of depth of sheetpiles criteria in different barrages

Sl. No.	Name of Barrage	Criteria assumed		Provision made		Remarks
		U/s sheet pile	D/s sheet pile	U/s sheet pile	D/s sheet pile	
1.	Dakpathar					
	a) Undersluices	R	1.25R	1.4R	2R	Levels as per assumed criteria come much near to floor in upstream and downstream. In d/s the cutoff depth has been fixed for safe exit gradient.
	b) Other bays	R	1.25R	1.5R	2.4R	As above.
2.	Ashan	R	1.65R	1.8R	-	No reason given for extra provision
3.	Ramganga					
	a) Undersluices	1.1R	1.25R	1.15R	1.18R	D/s sheet pile is above, than required due to presence of clay layer.
	b) Other bays	1.1 R	1.25R	1.1R	1.4R	D/s sheet pile kept lower than required to keep downstream sheet piles in undersluices and other bays at same level.
4.	Kosi					
	a) Undersluices	R	1.25R	1.3R	1.4R	The sheet piles have been taken to lower levels based on the criteria that the depth of up-
	b) Other bays	R	R	1.4R	1.42R	stream sheet piles below the floor should be at least equal to head over the floor and downstream pile at least 9.15 m below floor

(continued)

5.	Narora					
	a) Undersluices	R	1.15R	1.48R	1.15R	Upstream pile in undersluice bays has been taken to a lower level due to presence of clay layer. The downstream sheet piles in the undersluice bays have been kept higher to avoid penetration into clay layer.
	b) Other bays	R	1.15R	R	1.15R	
6.	Sone					
	a) Under-sluices	i) 1.25R with-out concen-tration of flow; ii) R with 20% concentra-tion of flow.		1.25R	1.35R	Upstream sheet pile has been provided on the consideration that depth below maximum water level should not be less than 1.25R. Down-stream sheet pile hasbeen provided from safe exit gradient considerations.
	b) Other bays	Do. ----				Upstream pile has been provided on the con-sideration that depth below apron should not be less than the maximum head acting on the barrage. Depth below retrogressed bed-level should not be less than the head above retrogressed bed-level.

scour depth. In any barrage or weir, the intensity of discharge will be different for the river bays and the undersluices and should, therefore, be taken separately for each.

One method of determining the silt factor is by the Lacey's slope formula:

$$S = \frac{f^{5/3}}{3340\ Q^{1/6}}$$ where S is the slope and Q the discharge in cumecs. This method may be approximate and can at best be taken for preliminary design.

Another method of determining the silt factor f is from the formula $f = 1.76\sqrt{d_m}$ where d_m is the average particle size in millimetres. This method is widely used in all major structures by determining the grain size diameter. For this purpose, material from several bore holes is analysed. The strata that is generally considered in this regard is upto the anticipated scour depth.

In using the Lacey's formula for scour depth, it is of utmost importance that the silt factor is evaluated accurately for a correct determination of the scour depth. The designs of the various structural parts of the barrage or weir, are mostly determined from scour considerations.

6.3.4. Length and thickness of loose aprons

6.3.4.1. In erodible river, launching aprons are provided in the vicinity of structures for their protection. The ~~material~~ natural river section is subjected to scouring, resulting in launching of the apron, which adapts itself to the scoured river geometry and serves as a transition between the constricted sec-

-tion at the structure and the natural river section. In a barrage or weir, such aprons are provided both on upstream and downstream of the pucca floor and also along the guide banks. The normal scour in the river bed takes place in the un-protected reaches of the river bed and progresses to the apron which launches. The material in launching apron, adopts itself generally to the scour pattern, thereby preventing deep scour in the immediate vicinity of the structure. To achieve this object, the size of the material in launching apron should be such that it is not washed away by the river current beyond the zone of its protective usefulness.

The existence of a general flow pattern and the dependence of the stone movements on their size, it can be possible to find, for each barrage or weir, a size of block which could not be moved by the water.(7)

6.3.4.2. Size of the blocks:

The problem of working out safe size of blocks or stones which will not be washed away by flowing water was attempted by Groat. Basically, the drag force has been equated to the frictional resistance of a block resting on sand.(10)

Tractive force (drag) exerted upon it by the flowing water $= T = \gamma.c.a.h$

Resisting force $= \mu.N.$

where, γ = Sp. wt. of water

c = form traction coefficient

a = sectional area of the stone

h = velocity head $= V^2/2g$

N = buoyant weight of block

μ = coefficient of sliding friction

If d be the side of a rectangular stone, then, $a = d^2$

volume $= d^3$

If γ_s = sp. wt. of stone, then

$$N = (\gamma_s - \gamma) d^3$$

$$\therefore \gamma . c . d^2 . h = \mu \, d^3 \, (\gamma_s - \gamma)$$

$$\therefore \frac{d}{h} = \frac{c}{(s-1)\,\mu}$$

where s = sp. gravity of stone. From field tests, Groat obtained the following values:

	Roughly rectangular stones	Rounded Boulders
Coefficient of sliding μ =	0.20	0.20
Coefficient of traction c =	0.73	0.68
c = d/h	3.04	2.83

Another attempt made in regard to determination of stone size is by S. V. Isbach.

Here he uses W. Airy's formula -

$$V_{min} = E_1 \, N \sqrt{d}$$

in which V_{min} isthe velocity required to move a stone falling upon, and rolling over, a fill;

$$N = \sqrt{2g \, \frac{s - \gamma}{\gamma}}$$

E_1 is a specific constant to be determined by experiment and as confirmed by practice Isbach finds $E_1 = 0.86$.

Isbach has also given a second formula

$$V_{max} = E_2 N \sqrt{d}$$

where V_{max} is the velocity requiredto dislodge a stone which has already found a 'seat' in the filling. In this case, the stone is partly protected by other stones and therefore a greater force is required to start it moving. According to Isbach $E_2 = 1.20$.

Isbach's method, rather than that of Groat, was generally confirmed by Hugh J. Casey, in connection with some rockfill dam designs for tidal-power project in U.S.A.

6.3.4.3. The current practice in this country in providing protection against scour for the apron, is:

(i) Downstream: Just after the end of the concrete floor an inverted filter 1.5 to 2.D (where Dis as defined below) long is provided. This generally consists of 0.6 m of graded filter material with 1.0 to 1.2 m deep concrete blocks with open joints over it. The openings between the blocks are filled with clean 'bajri' and gravel. Beyond the inverted filter the loose apron is provided.

(ii) Upstream: Just upstream of the concrete floor of the barrage or weir, a block protection of 0.6 m thick concrete blocks over 0.85 m of packed stone should be provided for a length equal to D. Upstream of the block protection, loose apron is provided in the same way as for the downstream apron.

The value of D can be determined from

$$D = xR - (\text{High flood level} - \text{floor level})$$
$$= xR - y$$

where x is the multiplier given in the following table and R is the depth of scour below maximum water level given by

$$R = 1.36 \left| \frac{q^2}{f} \right|^{1/3}$$

The value of R is calculated on the normal discharge per metre width at the section concerned, without allowing for any concentration of flow. If the usual concentration is allowed, the above coefficients haveto be reduced correspondingly.

Depth of scour for design of aprons:

Locality	Range	Mean
1. Upstream of pucca floor	1.25 R to 1.75 R	1.5 R
2. Downstream of pucca floor	1.75 R to 2.25 R	2.0 R

The upstream block protection and the downstream filter area are meant to be immovable. They are flexible and are supposed to adjust themselves to slight subsidence but they are not intended to fall in the same way as the loose aprons. Whenever these protections are damaged they should be made good at once. Their existence, in-tact, will be a definite safeguard against any damage to pucca floor.

The size of the blocks in most of the barrages and weirs is assumed arbitrarily. In the Girija Barrage, however, these have been calculated by the Isbach's formula.

The launching apron or the loose stone apron adjacent to the block protection as actually provided so that it may launch when the bed is scoured. Loose stone pitching has got an advantage, that it can spread uniformly over the scour slope during launching. As compared to this, cement concrete blocks of even 0.6 m x 0.6 m x 0.6 m size may not roll down easily during launching and will have a tendency of settling down vertically rather than covering the slope with uniform thickness. It is rather doubtful that c.c. blocks of above size will cover the entire slope fully and uniformly after launching. The possibility of irregular launching leaving large gaps on the slope of scour hole unprotected, cannot, therefore, be ruled out. From this consideration, loose stone pitching may be considered superior to c.c. blocks for use in apron material.

The detailed design of launching apron is illustrated under the design of guide banks.

Tables 6.2 and 6.3 give the details of block and loose stone protections as provided in existing barrages and weirs.

6.4. Design for sub-surface flow

The design of sub-surface flow is considered in the two following aspects:

a) Uplift pressure of the percolating water acting on the bottom of the floor;

b) The exit gradient and hence safety of structure against piping.

6.4.1. Uplift pressures

The floor of the barrage has to be safe against uplift pressures.

Table 6.2 - Statement of upstream pervious protection provided on various barrages

Name of Barrage	U/s loose stone protection		U/s flexible block protection		% Quantity of Block to Loose protections
	Length (m)	Thickness (m)	Length (m)	Thickness (m)	
1	2	3	4	5	6
Narora	19.2	1.22 m Loose stone	6.32	1.22 m (1.52x1.52x0.91 m C.C. blocks over 0.31 m stone)	33%
Harike	15.25	1.52 m Loose stone	9.10	1.82 m (1.52x1.22x0.91 m C.C. blocks over 0.61 Loose stone)	72%
Kosi	12.25	1.52 m Loose stone	7.60	1.52 m (1.52x1.52x0.91 C.C. blocks over 0.61 m Loose stone)	62.5%
Sone	18.20	1.52 m Loose stone	12.35	1.52 m (1.52x1.52x0.91 C.C. blocks over 0.61 m Loose stone)	69%
Gandak	11.0	1.81 m (1.52x 1.52x0.91 m blocks oveer 0.91 m stone)	6.4	1.82 m (0.91x1.22x1.22 C.C. blocks over 0.61 m stone)	58.5%
Wazirabad	12.25	1.52 m Loose stone	6.4	1.52 m (1.52x0.91x0.91 C.C. blocks over 0.61 m Loose stone)	52.5%
Yamuna Barrage Delhi	12.25	1.52 m Loose stone	7.32	1.52 m (1.52x0.91x0.91 C.C. blocks over 0.61 m Loose stone)	60%
Durgapur	10.7	1.22 m Loose stone	7.32	2.44x1.22x0.91 m C.C. blocks over 0.61 m Loose stone	85.5%
Girija	19.7	1.8 m Loose stone	12.87	2.13x1.52x0.91 C.C. blocks over 0.3 m Loose stone	50.7%

Table 6.3 - Statement showing provisions of downstream flexible block protection in different barrages

Name of Barrage	D/s Loose Stone protection		D/s flexible block protection				Flexible protection as % of Loose stone	Remarks
			Filter portion		Over Loose Stone			
	Length (m)	Thickness (m)	Length (m)	Thickness (m)	Length (m)	Thickness (m)		
1	2	3	4	5	6	7	8	9
Kosi	15.25	1.83 (av) Loose stone	16.45	1.68 0.91x0.91x0.91 m C.C.blocks over 0.76 m filter	-	-	98%	Designed by C.W. & P.C.
Gandak	11.00	1.83	7.0	1.83 0.91x0.91x1.22 m C.C.blocks over 0.61 m filter	5.8	1.83 0.91x0.91x1.22 m C.C. blocks over 0.61 stone	117%	Do.
Wazira-bad	15.2	1.52 riprap protection	6.0	1.52 0.91 m blocks over 0.62 filter	6.4	1.52 0.91 block over 0.61 stone	81%	Do.
Yamuna at Delhi	16.75	1.37 stone	6.85	1.37 0.91 blocks over 0.46 filter		1.37 0.91 blocks + 0.46 stone	84.5%	Do.
Narora	25.4	1.22 Loose stone	9.55	2.14 0.91 blocks over 1.23 filter	-	-	38%	U.P. Project
Ramganga	20.0	1.6 m Loose stone	11.1	1.6 1 m block over 0.6 m filter	-	-	55.5%	Do.

(continued)

1	2	3	4	5	6	7	8	9
Harike	22.9	1.52 Loose stone	6.0	2.14 1.22x1.22x0.91 blocks over 1.23 m filter	9.15	1.52 1.52x1.22x0.91 blocks over 0.61 stone	75.5%	Punjab Project.
Durgapur	31.2	1.52 Loose stone	14.5	1.83 1.52x1.52x1.22 C.C. blocks over 0.61 filter	8.55	1.83 1.52x1.52x1.22 C.C. blocks over 0.61 stone	88.5%	D.V.C. Project
Dakpathar	10.4	2.44x2.44x1.22 C.C. blocks	10.5	2.44x2.44x1.22 C.C. blocks over 1.22 m filter	-	-	102%	U.P. Project
Ashan	5.85	1.8x1.8x0.9 m C.C. blocks	7.95	1.8x1.8x0.9 m C.C. blocks over 1 m thick filter	-	-	136%	
Sone	27.4	1.52 m stone	12.9	1.52x1.52x0.9 m blocks over 0.61 m filter	12.5	1.52x1.52x0.91 over 0.61 m stone	92%	Bihar

6.4.1.1. It has been stated at Chapter 4, that the 'method of independent variables' evolved by Dr. Khosla is mostly used in view of its simplicity and accuracy for determining hydraulic gradient line for use in designing hydraulic structures.

In this method, a complex weir section issplit up into a number of elementary standard forms. Each elementary form is then treated as independent of the other. The pressures at the key points, i.e. the junction points of the floor and the pile line of that particular elementary form and the bottom point of that pile line (or at the bottom of the floor at change of elevation of the floor, i.e. when depressed), are then determined from the following equations given by Khosla:

i) Floor with pile not at end:

The percentage pressure $\emptyset$ at the key points is given by

$$\emptyset_E = \frac{1}{\pi} \cos^{-1} \left(\frac{\lambda - 1}{\lambda}\right)$$

$$\emptyset_C = \frac{1}{\pi} \cos^{-1} \left(\frac{\lambda_1 + 1}{\lambda}\right)$$

$$\emptyset_D = \frac{1}{\pi} \cos^{-1} \left(\frac{\lambda_1}{\lambda}\right)$$

where,

$$\lambda = \frac{\sqrt{1+\alpha_1^2} + \sqrt{1+\alpha_2^2}}{2}$$

$$\lambda_1 = \frac{\sqrt{1+\alpha_1^2} - \sqrt{1+\alpha_2^2}}{2}$$

and $\alpha_1 = \frac{b_1}{d}$; and $\alpha_2 = \frac{b_2}{d}$

ii) Floor with pile line at end:

$$\phi_E = \frac{1}{\pi} \cos^{-1} \left(\frac{\lambda-2}{\lambda}\right)$$

$$\phi_D = \frac{1}{\pi} \cos^{-1} \left(\frac{\lambda-1}{\lambda}\right)$$

where $\lambda = \dfrac{1+\sqrt{1+\alpha_1^2}}{2}$ since $\alpha_2 = \dfrac{b_2}{d} = 0$

These equations are applicable eitherto a pile at the upstream end of the floor or at the downstream end of the floor, since the stream-lines and pressure-lines will remain un-altered under a work, whether the head is applied from the upstream or downstream. The only change will be in the values to be assigned to the equi-pressure lines.

If $\phi_c = \dfrac{P_c}{H} \times 100$

where,

P_c = the residual head

H = total head;

be the percentage pressure at any point, then with the flow reversed the percentage pressure at this point will become $100 - \phi_c = (1 - \frac{P_c}{H}) \times 100$. In other words, an equi-pressure line which indicated 5% with normal flow will indicate (100-5) = 95% with reversed flow.

Dr. Kohsla has prepared curves for determination of the pressures based on the above equations which have been accepted to be correct by the Indian and European authorities as well

(Leliavsky and Harr)[37] These, therefore, can be used to save time.

(8)

The readings at the junction points are then corrected for

a) the mutual inteference of piles

b) the floor thickness

c) the slope of the floor

6.4.1.2. Mutual interferenceof piles:

This is given by the simple formula:

$$C = 19 \sqrt{\frac{D}{b_1}} \times \frac{d + D}{b}$$

where, C = the correction to be applied as percentage of head;

b_1 = distance between the piles;

D = the depth of pile whose influence has to be determined on the neighbouring pile of depth (d);

d = depth of pile on which the effect of pile (D) is sought to be determined; and

b = the total floor length.

This correction (C) is additive for points in the rear or back-water and ~~sub~~-subtractive for points forward in the direction of flow. This equation gives results within $2\frac{1}{2}\%$ of those obtained by experiments.

This equation does not apply to the effect of an outer pile on an intermediate pile o if the latter is equal to or smaller than the former and is at a distance less than twice the length of the outer pile. Subject to these limitations the above equation can be applied to find the influence of outer piles on intermediate ones and vice versa.

The mutual influence of piles is local. It mainly extends to a distance equal to the depth of the pile beyond which it gradually falls off till the residual effect at twice that distance is negligible in most cases.

6.4.1.3. Correction for floor thickness:

In the standard forms with vertical cutoff, the thickness of the floor is assumed to be negligible. Thus as observed from the curves, the pressures at the junction points E and C pertain to the level at the top of the floor. The pressures at the actual point E and C are interpolated by assuming straight line variation from the hypothetical point e to D and also from D to c.

6.4.1.4. Slope correction:

A suitable percentage correction is to be applied for a sloping floor. It has been established by Malhotra that the percentage of pressure under a sloping down or sloping up in the direction of flow are respectively greater or less than those under a horizontal floor for the same base ratios. The correction, therefore, applied in the method of independent variables is <u>plus</u> for the down-slope and <u>minus</u> for the up-slope following the direction of flow. The values for various slopes can be obtained from curve prepared by Malhotra (Plate X.1 of C.B.I. Pub. 12). The percentage correction given by the curve are then multiplied by the proportion of the horizontal length of slope, to the distance between the piles.

6.4.1.5. Sheet piles and pressure distribution:

The necessity of the piles in the upstream and downstream has been discussed in Para 6.5. The effect of the sheet piles on the distribution of the uplift pressures is also worth noting. The pressures under the downstream floor increase as the depths of the downstream pile line increases. The upstream pile line has little effect in reducing these pressures as spacing of these two is generally much more than the range of influence of either. The upstream end pile has little influence on the uplift pressures under the downstream floor. It will effect a reduction of pressures under the upstream floor, which is no consequence in the case of weirs or barrages where the load of water on the top of the upstream floor is much in excess of the uplift pressures.

Sometimes intermediate pile line is provided generally at the toe of the glacis or at the junctionof the upstream floor and the glacis. They neither prevent undermining of the floor at the upstream nor at the downstream, nor do they materially alter the pressure distribution to give less uplift pressures under the downstream floor. But they act as an important second line of defence so that even if the impervious floor is damaged at either the upstream end by failure of the end piles under abnormal scour, the rest of the floor and the superstructure will be saved from collapse by the intermediate piles.

Opinions however differ regarding the number of intermediate pile lines. But if an intermediate pile line is provided it should be at the toe of the glacis.

6.4.2. Exit gradient

The exit gradient downstream of weir or barrage has to be safe against piping.

6.4.2.1. Weir failure from seepage flow can occur by -

(a) undermining of the sub-soil;

(b) uplift due to pressure under the floor being in excess of the weight on the floor.

The failure by undermining is the most common, so that a knowledge of its causes and measures to prevent it, are of importance in design.

The undermining of the sub-soil starts from the tail-end of the work. It begins at the surface due to the residual force of seepage water at this end being in excess of the restraining forces of the sub-soil which tend to hold the lateer in position. Once the surface is disturbed, the dislocation of sub-soil particles works further down and, if progressive, leads to the formation of cavities, below the floor into which the latter may collapse. According to the commonly accepted ideas, this undermining is supposed to result from what is known as 'piping', that is, the erosion of sub-soil by the high velocities of flow of water through it. Water has a certain residual force at each point along its flow through the sub-soil, which acts in the direction of flow and is proportional to the pressure-gradient at that point. At the tail-end this force is obviously upwards and will tend to lift up the soil particles, if it is more than the submerged weight of the latter. Once the surface particles are disturbed, the resistance against upward pressure of water will be further reduced, tending to progressive disruption of the sub-soils.

The gradient of pressure at which this occurs has been called first by Terzaghi as 'Flotation gradient', later by Haigh as 'Bursting gradient' and as 'exit gradient' by Khosla.

By considering an elementary cylindrical element on the stream line of the flownet it can be shown that the exit gradient is equal to $(1-n)(G-1)$ where G = sp. gravity of soil grains, n = porosity of the soil. When this gradient equals the value given by the above equation, it is called the 'critical gradient'. If critical gradient is reached, failure by undermining is imminent.

The value of G for most river sands is very nearly 2.65 and an average value of porosity is 0.4. The exit gradient will then be

$$(1-0.4)(2.65-1) = 0.99 \text{ or, say, } \quad 1.0$$

Hence for any structure, the exit gradient should be kept considerably lower than the critical gradient, by providing a suitable factor of safety. This takes care of the unfavourable conditions of sub-soil. The foundation soil may not be quite homogeneous. The packing and pore space may differ in places. There may be local intrusion of clay beds which deflect flow or there may be zones of very porous material which may induce concentration of flow from all around. To account for all these uncertainties in nature, a factor of safety has to be applied to the critical exit gradient to obtain a safe value.[8] These are:

Shingle	4 to 5
Coarse sand	5 to 6
Fine sand	6 to 7

For an apron length b, with a vertical cutoff at the end of depth d, the exit gradient is given by the equation

$$G_E = \frac{H}{d} \cdot \frac{1}{\pi\sqrt{\lambda}}$$

where,

G_E = the exit gradient

H = the maximum head of water

$$\lambda = \frac{1 + \sqrt{1 + \alpha^2}}{2}$$

and $\alpha = b/d$

If d = 0, it is obvious from the equation that G_E is infinite. Hence a cutoff should be provided at the downstream end.

6.4.2.2. The exit gradient for all the weirs and barrages are calculated by this formula for the floor length and depth of cutoff adopted. It can be seen that the formula is dependent on two parameters - the length of the floor and the depth of cutoff. These two since can be varied, therefore for a particular exit gradient a most economical design can be obtained by a proper combination of these two dimensions. The depth of the downstream cutoff, however, has to be minimum, from requirement of scour.

6.4.2.3. The necessity of providing a factor of safety has been indicated at para 6.4.2.1. However, if the exit is covered with an inverted filter, which retains the soil particles without interfering with the flow of water, non-uniform flow through such heteogeneities can be taken care of and the filter provides further safety against exit gradient. Therefore, the problem of eliminating the danger of piping can best be solved by providing an inverted filter of adequate length in areas where seepage water is likely to emerge downstream of a hydraulic structure. The presence of filter prevents surface erosion of the soil and undermining. Thus a large factor of safety is afforded.

This filter bed of layers of increasing permeability from bottom upwards is generally protected against dislocation due to the actionof surface flow by over-laying it with heavy blocks provided with open joints.

6.4.2.4. Table 6.4 gives the values of safe exit gradients as adopted in various existing barrages and weirs. The values of exit gradients have been calculated both at the downstream floor level and at the bottom of filter. The C.W. & P.C. practice is to calculate the exit gradient at the bottom of filter in all their designs. One of the arguments is that the filter may become faulty, resulting in improper function of the same. The C.B.I. publications, which furnishes the safe exit gradients recommended by Khosla, does not specify the level to which these safe values pertain. However, Khosla while working out the exit gradient for the Khanki Weir reckons the depth of

Table 6.4 - Exit gradient in different barrages and weirs

Sl.No.	Name of Barrage	Exit gradient			
		At d/s floor level		At bottom of filter	
		U/sluices	Other bays	U/sluices	Other bays
1.	Narora	1/6	1/6.075	1/4.55	1/4.25
2.	Ramganga	1/6.04	1/6.0	1/5.13	1/5.0
3.	Dakpathar	1/5	1/4.9	1/3.5	1/3.91
4.	Ashan	1/4	-	1/2.13	-
5.	Kosi (Hanuman-nagar)	1/6	1/6.15	1/5.3	1/5.5
6.	Ferozpur Weir	1/5	-	1/4.3	-
7.	Ferozpur Barrage	1/6.25	-	1/5.6	-
8.	Sukkur	1/5.1	-	1/4.1	-
9.	Trimmu	1/5.0	-	1/4.5	-
10.	Girija	1/5.5	1/4.95	1/5.5	1/4.95
11.	Salandi	-	1/5.9	-	1/5.2

pile from the downstream floor level.

6.4.2.5. Terzaghi[22], while dealing with the mechanics of piping, specifies that the factor of safety against failure by piping at the exit end can beincreased by loading the soil by an inverted filter.

It therefore appears quite reasonable that with the provision of a properly designed inverted filter weighed down by the blocks, the exit gradient can be safely calculated at the downstream floor level. However, the discretion should be left to the choice of the designer.

The C.W. & P.C. value therefore relies on higher factor of safety.

6.4.2.6. Designof filter:

The following limits are recommended to satisfy filter stability criteria and to provide ample increase in permeability between base and filter. These criteria are satisfactory for use with natural sand and gravel, or with crushed rock.[20]

1) $$\frac{D_{15} \text{ of the filter}}{D_{15} \text{ of the base material}} = 5 \text{ to } 40,$$

provided that the filter does not contain more than 5% of material finer than 0.074 mm.

2) $$\frac{D_{15} \text{ of the filter}}{D_{85} \text{ of base material}} = 5 \text{ or less}$$

3) $$\frac{D_{85} \text{ of the filter}}{\text{Max. opening of pipe drain}} = 2 \text{ or more}$$

4) The grain size curve of the filter should be roughly para-

llel to that of the base material.

The D_{15} is the size at which 15% of the total soil particles are smaller; the percentage is by weight as determined by mechanical analysis. The D_{85} size is that at which 85% of the total soil particles are smaller. If more than one filter layer is required, the same criteria are followed; the finer filter is considered as the 'base material' for selection of the gradation of the coarser filter.

In addition to the limiting ratios established for adequate filter design, the 75-mm (3-inch) particle-size should be the maximum utilised in a filter to minimise segregation and bridging of large particles during placement of filter materials.

The filter layers for coarse filter material (75-mm max. size) are usually not less than 20 cms (8 in.) in thickness and layers of finer filter materials are often of 15 cms (6 in.) minimum thickness.

Length of filter

At the downstream end of the concrete apron there should be a filter length to improve the exit gradient. The length of this filter, however, has been differently adopted by different authorities. The necessity of the filter has been explained by Terzaghi(22) as follows:

"By model tests it has been found that the rise of the sand occurs within a distance of about D/2 (where D is the depth of the downstream pile below the bed level). The failure therefore starts within a prism of sand having a depth D and width D/2.

"At the instant of failure the effective vertical pressure on any horizontal section through the prism is approximately equal to zero. Therefore, piping occurs as soon as the excess hydrostatic pressure on the base of the prism becomes equal to the effective weight of the overlying sand. If the factor of safety against failure by piping is too small, it may be increased by establishing on top of the prism oefa an inverted filter which has a weight." From this appears that a filter of length D/2 would suffice.

Dr. Khosla recommends that at the downstream end, there should be an area of inverted filter of length equal to 1.5D to 2D where,

$$D = xR - (\text{H.F.L.} - \text{Floor level})$$

x being the scour factor.

The Central Water & Power Commission generally, in their designs, provide approximate half the above length (i.e. D), though in some full length has been provided vide table

The U.P. Irrigation Research Institute has evolved a method of determining thefilter length as below:[35]

"It is assumed that (i) the effect of a cutoff, at the end of the floor, in reducing exit gradient is predominantly in a downstream pervious length equal to about the depth of the cutoff, (ii) At greater distances its value becomes almost constant irrespective of the depth of cutoff. Based on this, the criteria adopted in determining the filter length is that the exit gradient at the end of the filter length is half its value at the beginning, i.e. the factor of safety is twice at the end. Based on experimental data, they have prepared curves correlating b/d and f/b or f/d where , b = floor length

f = filter length

d = depth of cutoff

No application of this method appears to have been done so far. As an illustration we may apply this to the existing barrages:

i) Sone Barrage

a) Undersluice bays:

b = length of apron = 48 m

d = depth of cutoff = 7.32 m

$$\frac{b}{d} = 6.56$$

From the graph $\frac{f}{b} = 0.3$

$\therefore$ Length of filter = 48 × 0.3 = 14.4 m as against the provided length of 14.5 m.

b) Spillway bays:

b = 48 m

d = 5.95 m

$\frac{b}{d} = 8.05$

$\frac{f}{b}$ from graph = 0.25

$\therefore$ Length of filter f = 0.25 × 48 = 12 m. as against the provided length of 12.85 m.

ii. Ramganga

ii) Ramganga

a) Undersluices:

$b = 63.31$ m

$d = 6.15$ m

$\frac{b}{d} = \frac{63.31}{6.15} = 10.3$

$\frac{f}{b} = 0.225$

$f = 63.31 \times 0.225 = 14.22$ m as against 11.10 m actually provided.

b) Spillway bays:

$b = 50.25$ m

$d = 5.75$ m

$\frac{b}{d} = 8.75$

$\frac{f}{b} = 0.25$

$f = 50.25 \times 0.25 = 12.5$ as against 7.8 m actually provided.

Hence in both cases the provision is less.

iii) Salandi Barrage

$b = 51.30$ m

$d = 4.87$ m

$\frac{b}{d} = 10.52$

$\frac{f}{b} = 0.22$

$f = 51.3 \times 0.22 = = 11.28$ m as against 9.45 m actually provided.

This indicates that the length of the filter by this method is slightly on the higher side than provided.

6.5. Specific problems

A raising of the cistern level by 0.61 m was allowed in the Kemri Barrage by providing ancillary works. This was necessitated by scour considerations as can be seen from the model experiments conducted at the U.P. Irrigation Research Institute (14) and discussed below.

The new Kemri Barrage is located 300 m upstream of the existing old Kemri spillway. The barrage water-way was designed for a maximum discharge of 1,415 cumecs giving a tightness factor of 0.8 4 over Lacey's stable water-way. The cutoffs were designed for a maximum discharge of 2,126 cumecs assuming f = 1. The barrage consists of 15 bays of 9.15 m wide each. The two end-bays at either end comprise the undersluices and have been separated from the barrage by two divide walls, while the remain-portion of barrage has been sub-divided by another two divide walls into three compartments consisting of three, five and three bays, respectively. The downstream floor is kept at R.L. 178.0 for the undersluice bays while the barrage bays are kept at R.L. 178.31.

When the profile for the undersluices was tested for a discharge of 494 cumecs corresponding to the maximum discharge, scouring was observed in the model at a distance of 18.3 m from the end of the downstream floor. The main cause for excessive scour appears to be that the jump was submerged. The cistern level varied from R.L. 178.00 to R.L. 180.00 for discharges ranging from 42.5 cumecs to 457.5 cumecs, d_2 varied from 1.65 m to

4.05 m. The submergence was reduced with advantage by suitably raising the cistern level with the river bed level at R.L. 179.80. The model was then run with downstream floor raised to R.L. 178.62, i.e. 15% of d_2. The bed scour was again observed at a distance of 18.3 m, but somewhat less. The change in glacis slope from 2:1 to 3:1 showed an improvement in the formation of jump and reduction in the bed scour.

An attempt was made to normalise the flow beyond the downstream pucca floor and see the effect in the bed scour by provision of a suitable dentated sill, sited at the end of the floor. A 0.91 m high sill ($0.226d_2$) further reduced the bed scour. This when supplemented by a row of floor blocks 1.22m high and sited at a distance of 4.26 m from the toe of glacis, further reduced scour. The cistern level of the undersluice was therefore fixed at 178.62.

On a similar reasoning, the barrage floor was also raised to 178.93.

Chapter 7

STRUCTURAL DESIGN OF BARRAGES AND WEIRS

7.1. The downstream apron of a weir or barrage has to be designed to withstand two forces:

i) the uplift pressures acting underneath the floor;

ii) the maximum unbalanced head caused by the hydraulic jump.

The latter aspect is dealt separately.

From considerations of either of the two, the floor can be designed as

i) a purely gravity floor,

or ii) a reinforced concrete mat.

The economics of these two designs are dealt with at para 7.4.

7.2.1. For a gravity floor once the uplift pressures are determined, the floor thickness at any section can be calculated by dividing the residual head at that section divided by the submerged weight of concrete or masonry. The maximum head acting across the structure will then be when the water on the upstream is ponded upto its maximumlevel with no water on the downstream. The uplift pressure at any section is obtained by assuming a linear variation of hydraulic gradient line between the pile lines.

A question arises here in respect of the downstream reference level for accounting the maximum head. The downstream bed level has generally been assumed in some cases, while in other cases the downstream floor level. The sill level or bed level (not retrogressed bed level below the floor) should be taken into account to worst condition.

The Central Water & Power Commission assumes low tail-water level. They also assume presence of water in the cistern to oppose the residual head to arrive at the thickness. This, however, may not be correct as the tail-water level may be at or below the top of sill level, or the gates maybe fully sealed and/or the floor may be under repair. Also, this water depth may not be available at all times.

7.2.2. A point that needs consideration in design is regarding the effective pressure that should be taken to act on the underside of the floor, is the full indicated pressure or only a fraction of it. One school of thought considers that as the water and soil cannot be in contact with the underside of the floor at one and same time, the full indicated pressure will act only on that part of the surface which is not in contact with the soil. The other school contends that water will exert full indicated pressure on the entire area directly or through the soil grains as the case may be. The problem is similar to that of retaining walls with saturated fills.

The general opinion[8] in this case is to take the full uplift pressure. This assumption appears to be sound, as local settlement of the sub-soil may result at certains parts of the floor, where the entire area will be in direct contact with the water.

7.2.3. In the design of a gravity floor no tension should be permitted in the concrete. In the Nangal Barrage[25] the gravity floor is assumed to span between the dummy piers and the floor thickness is evaluated by assuming that the uplift pressure is balanced partly by the floor thickness and partly by the dummy piers. Tension has been taken into account. This, however, doesnot appear to be sound practice for design of a gravity floor.

Further, on the top of the floor a wearing coat (about 15 cm) with reinforcement, at cistern level is provided to ensure availability of total thickness of floor. Damage to wearing coat is continually made up at time of annual repairs. This generally is not accounted for in the total thickness of the floor specially in boulder reaches. However, in some cases, as at Narora Barrage, part of the wearing coat was taken into account in the design calculations.

7.3. Thickness of floor from jump consideration

The hydraulic jump introduces a significant factor in the design of downstream floor of a barrage or weir, which may cause its failure, though the floor may be safe from undermining or against uplift pressures for normal ponding condition.

The uplift pressures, at any point of the foundation of the weir or barrage due to sub-surface flow, are given by the vertical intercepts between the pressure gradient lines and the point under consideration. As stated earlier, maximum uplift pressures are normally imposed on the work when the water is ponded upto the highest level on the upstream side without any discharge passing downstream, and total head created by the ponding becomes operative.

When a certain discharge is passing on the work and a hydraulic jump is formed, the seepage head, i.e. the difference between the water level upstream and downstream is less than the seepage head with no flow.

From a reference to the section of the Trimmu Weir[8] (Fig. 7.1), it can be seen that the net uplift pressures for maximum flow, will be very small, being the ordinate between the hydraulic gradient line and the water surface level except in the trough where the vertical ordinate was found to be as much as 3.42 m. The downstream floor at the point of formation of hydraulic jump has to withstand this unbalanced pressure.

The standing wave, however, is not stationary and in some cases can move downstream with the retrogression of levels and reduction in intensity of discharge, when the floor has to withstand the unbalanced heads. For the structure to be safe, it is advisable that the hydraulic jump should be made to form on the glacis rather than on the floor, even with a retrogressed bed level.

It can further be seen that the maximum unbalanced pressures will be at the deepest point of the trough and will be smaller on either side. Hence, if the floor at the end of glacis is designed for the maximum ordinate the thickness of the floor will be too great which will be wasteful. It is, therefore, desirable that some allowance should be made for the distribution of the pressures, due to somewhat oscillating nature of jump, for the thickness of the floor. Khosla recommends that the ordinate of unbalanced head can be taken as two-thirds. For the Nangal Barrage, the ordinate has been taken as three-fourths.

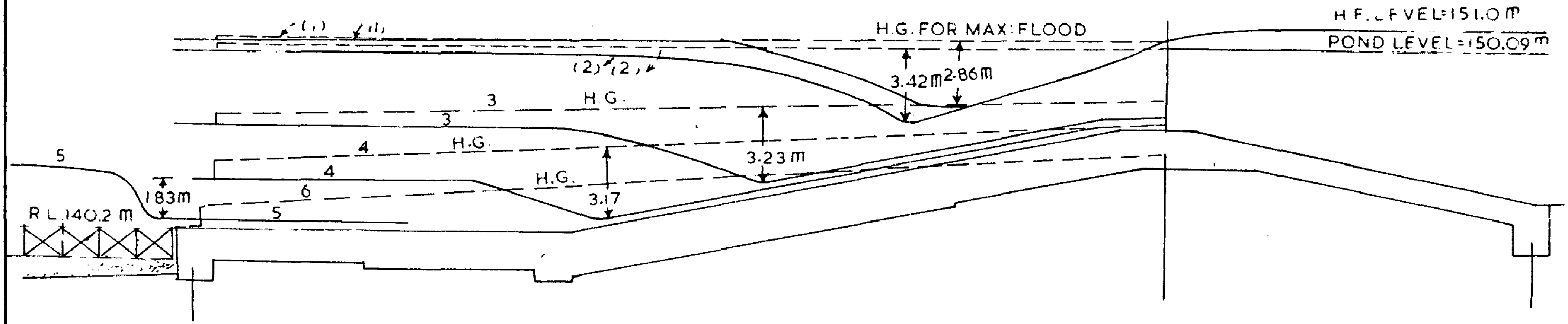


FIG. 7.1 SHOWING PROFILES OF STANDING WAVES AND HYDRAULIC GRADIENT LINES FOR MAXIMUM FLOOD AND LOWER DISCHARGE (TRIMMU WEIR)

The Central Water & Power Commission assume as $\frac{1}{2}(D_2 - D_1) + \phi.H_L$ where ϕ is the percentage of pressure at the location of the jump. This factor, however, is at the discretion of the engineer.

The final thickness of the downstream floor has to be designed for the maximum of either unbalanced uplift pressures when hydraulic jump is formed after accounting for their distribution at the trough, or the uplift pressures due to sub-surface flow for maximum ponding and no tail-water level.

In calculating the unbalanced head, the significant factor which needs attention is the profile of the water surface between d_1 and d_2. This can be determined for different discharges from the basic equations of the hydraulic jump for various positions of the glacis. The curve can then be plotted. But the fact remains that this theoretical profile may change when appurtenances are added to the basin. Hence these will be approximations, if the profiles are not taken from model experiments.

7.4. Considerations in design of an R.C.C. raft as an alternative to gravity floor and basis of design

It was stated earlier at Para 7.1, that the barrage floor can be designed as an R.C.C. raft as against a gravity floor. The main consideration in this regard is economics. Sometimes construction difficulty in dewatering the foundations to level dictated by concrete gravity floor may dictate choice of an R.C.C. raft.

The additional cost of reinforcement and richer concrete has to be compared mainly with saving in cost involved in the excavation and dewatering of foundation, for a gravity floor, and to minor extent savings in the base width of abutments and consequent reduction of floor length between them; and the height of the piers and saving in foundations of piers and abutments, as they would rest on the R.C.C. floor; savings in copper seal joints around piers and abutments, if proposed.

One aspect, however, needs consideration. The progress of concreting generally retards with the presence of reinforcement. R.C.C. rafts are seldom cheaper than gravity floors. But these have, sometimes, to be provided.

7.5. Theory of Raft design

7.5.1. The analysis and computation of stresses in the barrage rafts, as per current practice, are based on various prevalent methods of analysis of elastic foundation. These methods are usually developed on the assumption that the reaction forces of the foundation are proportional to the settlement of the same and are, hence, proportional at every point, to the deflection of the beam at that point. If y_1 is the deflection at any point then the soil reaction is given by $K_s \times y_1$ where K_s is the modulus of sub-grade reaction.

Terzaghi[26] defines the sub-grade reaction as the pressure p, per unit area of the surface of contact between a loaded beam or slab and the sub-grade on which it rests and onto which it transfers the loads. The coefficient (or modulus) of sub-grade reaction K_s is the ratio between this pressure at any given point of the surface of contact and the settlement y produced by the load application at that point:

$$K_s = \frac{p}{y}$$

The value of K_s depends on the elastic properties of the sub-grade and on the dimensions of the area acted upon by the sub-grade reaction.

7.5.2. There are two methods of analysis of elastic foundations: one as advanced by Hetenyi[27] and the other by Baker.[28]

The basic differential equation of deflection for a beam is given by

$$EI \frac{d^4y}{dx^4} = -Ky$$

where E is the modulus of elasticity of material of the beam and I, the moment of inertia of the beam section.

A general solution of the equation is given by

$$y = e^{\lambda x} (C_1 \cos \lambda x + C_2 \sin \lambda x) + e^{-\lambda x}(C_3 \cos\lambda x + C_4 \sin \lambda x)$$

where,

$$\lambda = \sqrt[4]{\frac{K}{4EI}}$$

By deriving the values of the various constants in the above equation, Hetenyi finally furnishes the following equations for a beam of infinite length resting on elastic foundations:

a) Under a transverse load P (fig 7.2)

$$y = \frac{P\lambda}{2K} e^{-\lambda x} (\cos \lambda x + \sin \lambda x)$$

$$\theta = -\frac{P\lambda^2}{K} e^{-\lambda x} \sin \lambda x$$

$$M = \frac{P}{4\lambda} e^{-\lambda x} (\text{Cos } \lambda x - \text{Sin } \lambda x)$$

$$Q = -\frac{P}{2} e^{-\lambda x} \text{Cos } \lambda x$$

b) Under a moment M_o

(fig 7.3)

$$y = \frac{M_o \lambda^2}{K} \cdot B_{\lambda x}$$

$$\theta = \frac{M_o \lambda^3}{K} \cdot C_{\lambda x}$$

$$M = \frac{M_o}{2} \cdot D_{\lambda x}$$

$$Q = -\frac{M_o \lambda}{2} \cdot A_{\lambda x}$$

where,

$$A_{\lambda x} = e^{-\lambda x} (\text{Cos } \lambda x + \text{Sin } \lambda x)$$

$$B_{\lambda x} = e^{-\lambda x} \text{Sin } \lambda x$$

$$C_{\lambda x} = e^{-\lambda x} (\text{Cos } \lambda x - \text{Sin } \lambda x)$$

$$D_{\lambda x} = e^{-\lambda x} \text{Cos } \lambda x$$

Here y is the deflection, θ the slope, M, the bending moment and Q the shear force.

These formulae are extensively used in the computation of the stresses in a barrage raft. The coefficients A, B, C and D are functions of λx and can be readily obtained from tables furnished by Hetenyi once the coefficient of sub-grade reaction is known.

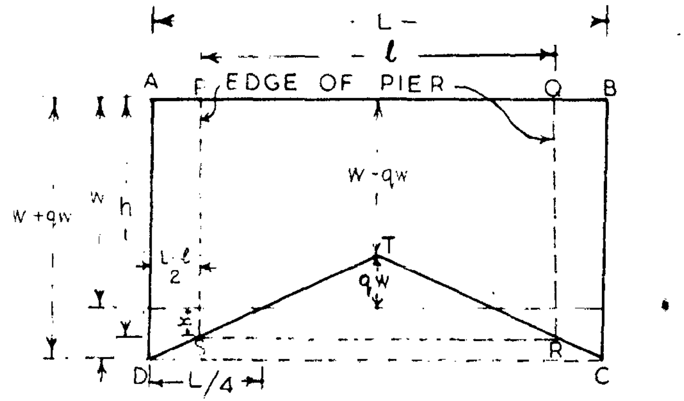


FIG. 7.4 VARIATION OF SOIL REACTION .

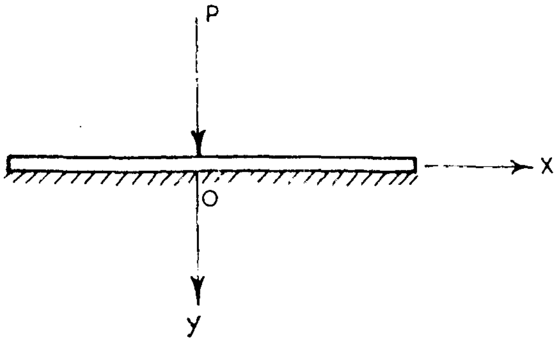


FIG. 7.2

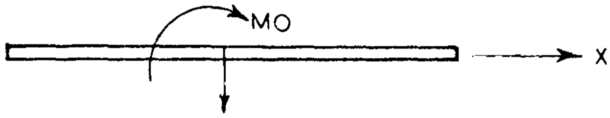


FIG. 7.3

7.5.3. According to Baker's method the variation of soil reaction between the piers and the centre is taken to be straight line and therefore can be expressed as (W+qw) at the end span and (W-qw) at the middle, where W is the average soil reaction, q being variation constant of soil reaction. W consists of average value of soil reaction due to the loads transmitted through piers and uniform loads like uplift pressures &c. The soil reaction varies from maximum at the end, to the minimum at the centre of the span shown in figure.(7·4)

Let y_1 = deflection due to the average soil reaction W;

y_2 = deflection due to the varying element of qw

$$y = \text{net deflection} = y_1 + y_2 \qquad \text{(i)}$$

Equation (i) expresses the characteristics of the beam and is called beam line equation.

The soil reaction below the raft at the end is W + qw and theerefore the settlement at the end point is $\frac{W + qw}{K}$ and at the centre of the span $\frac{W - qw}{K}$. Therefore, the difference in settlement between the pier point and the centre

$$= \frac{W + qw}{K} - \frac{W - qw}{K} - \frac{2qw}{K} \qquad \text{(ii)}$$

Equation (ii) expresses the characteristics of the soil and is called soil line equation. Solving (i) and (ii), the deflection, i.e. difference of settlement between centre and pier-point, can be found out.

Deflection y_1 for a fixed beam of span L for uniformly-distributed load W is $y_1 = \frac{WL^4}{384EI}$

Here $y_1 = \frac{(W + qw)\, L^4}{384\ EI}$

and difference y_2 for a beam of span L for triangular load is given by

$$y_2 = \frac{7}{1920}\ \frac{q\ WL^4}{EI}$$

The sign y_2 being negative, the net deflection

$$y = y_1 - y_2$$

$$= \frac{(W + qw)L^4}{384\ EI} - \frac{7\ qwL^4}{1920\ EI}$$

$$= \frac{WL^4}{384\ EI} - 0.0010429\, WL^4/EI$$

7.5.3.2. Load conversion factor:

The loads under the piers are assumed not to generate any bending moments and the uniformly-distributed loads being transmitted through the pier have got to be converted into the loads causing bending moments. In Fig. 7·4 the rectangle ABCD represents the load of one span being transmitted through the pier and rectangle PQRS minus Δ STR is the load causing bending moment. The ordinate AD will change into PS at the edge of the pier. The values can be worked out as below:

Let h be the ordinate at the edge of the pier and the load causing bending moment be WR; then

$$h = W + \frac{qw}{L/4} \cdot \frac{2\,l - L}{4}$$

$$= W + \frac{Wq(2\,l - L)}{L}$$

$$= W \left| L + \frac{q\ (2\,l - L)}{L} \right|$$

$$= W \left| 1 - \frac{q\ (L-2\ l)}{L} \right|$$

$$W_R \ = h.l \ = Wl \left| 1 - q(1 - \frac{2\ l}{L}) \right|$$

Hence load conversion factor for the rectangular portion PQRS

$$= \frac{Wl \ \left| 1 - q(1 - 2\ l/L) \right|}{WL}$$

where WL is the total load being transferred through one pier.

$$\therefore \text{ Conversion factor} = \frac{l}{L} \left| 1 - q\ (1 - \frac{2\ l}{L}) \right|$$

7.5.4. Loads and bending moments

These are considered as below.

(a) Live loads:

These consist of the live loads on the road bridge, gate bridge, wind loads. These are generally taken in accordance with the I.R.C. Codes.

(b) Dead loads:

These consist of the dead loads of the bridge decking, railing, gates, hoist, operating platform etc.

(c) Bending moments:

For calculating the bending moments, the raft is considered to be continuous structure with ends fixed. The end fixing is provided by the heavy load of the abutment and piers, which are constructed monolithic with the raft. The fixed-end moments are calculated based on the usual formula, depending on the type of loading.

Uplift:

The uplift plays an important part in the computation of bending moments. The existence of uplift reduces the effectiveness of vertical loads coming from above. This can be considered in the following ways:

i) Net uplift will be determined bysubtracting from the gross uplift, the submerged weight of the slab. This uplift load shall be deducted from the total live load and dead loads; the bending moment shall be due to the net load. The total bending moment shall be the bending moment due to the net uplift and the live loads etc.

ii) The uplift will be neglected and the bending moments are calculated for live and dead loads transmitted through the piers. The load of the slab will not generate any bending moment or relative deflection and hence neglected in the calculations.

The raft has to be designed for the maximum bending moment obtained from the above two considerations.

7.5.5. Reinforcement

Terzaghi recommends that reinforcement in the raft shall be increased by 100% to take care of any variation in soil condition and other uncertainties.

The I.S. Code recommends an increase of 50% only[29] for a raft designed on soil-line method.

In the Narora undersluices, the increase has been taken as 50% on the assumption that the variation and uncertainties in soil condition will be less, because the soil is practically uniform.

7.5.6. Evaluation of coeffficient of sub-grade reaction

Terzaghi[28] recommends the following procedure:

(a) General procedure:

The numerical values of the coefficients of sub-grade reaction K_s required for the solution of engineering problems can either be estimated on the basis of published observational data or, else, they can be derived from the results of field tests to be performed on the sub-grade of the proposed structure. For practical purposes, rough estimates of these values serve their purpose.

(b) Vertical sub-grade reaction:

As a basis for estimating the coefficient of sub-grade reaction K_s for beams and slabs, the value K_{s1} for a square plate with a width of 30 cms has been selected, because this value can, if necessary, be determined by averaging the results of several loading tests in the field, at the site of structure.

If the sub-grade consists of cohesionless or slightly cohesive sand K_s can be estimated on the basis of empirical values of K_{s1}. Such values are readily available in the I.S. Code in kg/cm^3 for various densities, i.e. loose, medium or dense corresponding to the state of the soil, i.e. dry or moist or submerged state. The density category of the sand can be ascertained by means of a standard penetration test.

Once the value of K_s to be used in the solution of a given problem can be computed. Experience has shown that the value K_{s1} for a beam with a width of 30 cm resting on sand is roughly equal to that of a square plate 30 cm wide. For a beam with a width of B cm or for a mat acted upon by K line loads spaced B cm; K_s is determined by the equation

$$K_s = K_{s1} \left\{\frac{B + 30}{2B}\right\}^2$$

Values of K_{s1} for clayed soils have also been specified in the code.

7.5.7. Two points need attention in the design of the raft by this procedure:

(i) The selection of K_s:

For an increase in the value of K_s , the maximum foundation pressure increases. There will be increase in bending moment. As already stated, Terzaghi recommends that for practical purposes rough estimates of the value would serve the purpose. Underestimation, if any, is therefore accounted for by 50-to-100% increase in reinforcement.

(ii) Another assumption made is the straight line variation between the load and the foundation settlement. This means a constant elasticity has been assumed. For dense sand, the stress reaches a maximum value at the comparatively low strains[30] and then decreases rapidly with increase of strain, the stress becomes more-or-less constant. Loose sand shows relatively slower rate of increase of stress with strain, the stress becomes maximum at comparatively large strains and afterwards it decreases very slowly. Hence by assuming linear variation the settlement may be under-estimated to a certain extent.

7.6 Pressure-relief arrangement:

Economy to a certain extent can be achieved in the downstream floor thickness if suitable pressure release arrangements can be introduced beneath the floor. This would consist of an inverted filter with G.I. pipes embedded into it. If a pressure-release arrangement is provided, obviously there would be reduction in pressure. But the extent to which this reduction should be affected, is yet unestablished. This aspect was studied by electrical analogy experiments by Sangal[36] who recommends 5% reduction if a filter is located at the toe of the downstream floor. With the present state of knowledge this cannot be taken for granted unless actually corroborated with field observations which are lacking.

However, there is a second school of thought regarding provision of filter, below the concrete apron, and consequent reduction in pressure. If allowance is made in design for reduced uplift-pressures due to drainage and if the system gets choked fully or partially, the floor has to stand pressures, much in excess of those for which it has been designed and may give way. If, on the other hand, some fault occurs in the filter and sand movement starts, the work may fail by undermining.[8]

It is customary with the Central Water & Power Commission, to provide filter beneath the floor without accounting for any reduction in pressure. It is treated as a second line of defence. Such filters are provided at the Mundali, Narora, Sone, Gandak Barrages.

7.7. Three dimensional effects

The problem of three dimensional seepage was investigated, at length, by Dr. Satish Chandra,[44] both by analytical and electrical analogy methods. The main causative factor in this problem is the elevation of the stable water table on the sides of the work. The flow of seepage water below the structure through the porous media is almost always three dimensional. However, the seepage can be considered 2-dimensional if the width of the floor is greater than the length of the floor, when most of the flow lines through the middle portion of the structure will be more-or-less in vertical planes for flow from upstream to downstream. Such a condition is encountered in case of weirs and barrages constructed across wide rivers, to divert water into canals. When the width of the floor is not too large as compared to the length of the floor, the seepage flow will take place through the porous media below the structure as well as adjacent to it and below the abutments of the structure. The flow under such situations will be markedly three dimensional. The water table will further modify the flow and its location and elevation will substantially alter the uplift pressures b below the floor.

The presence of the water table would have a significant effect on the potentials below the floor. The water head potentials along the abutments have been found to increase with rising water-table elevations. With the increase in the distance of the water table from the structure, the potentials decrease, if the water table is mid-way between upstream and downstream water level of the structure or lower, and increase for water-table elevation higher than this. With increasing length-width ratio, the potentials increase on the upstream and decrease on the downstream.

In barrages and weirs, for the design of the downstream floor which is actually controlled by uplift pressures, the possibility of the uplift pressures calculated by three-dimensional effect being less than those computed by two-dimensional seepage is rather low. Near the river, the water-table is seasonally likely to be higher.

For all the barrages constructed in Uttar Pradesh, tests have been made by electrical analogy method to study the effects of three-dimensional seepage. In all cases, it has been found that there was an increase in uplift pressures in the bays adjacent to the abutments.

Incidentally, boxing of foundations of the structure reduces these uplift pressures to a certain extent, though this is actually provided for stabilising the foundation soil. In all cases, the upstream sheet pile is extended parallel to the river flow, in the upstream direction for a distance equal to 1.5 to 2 times the pile depth, to reduce three-dimensional seepage effects.

7.8. Earthquake effects

For diversion structures located in seismic regions the design of the floor, as a gravity section, is not governed by forces due to earthquake. For super-structure, however, the earthquake effects have to be considered in accordance with the provisions of the I.R.C. Code of Practice. But since there would be an increase in dead load of the structure, the earthquake effects would be of importance when the floor is designed as an R.C.C. raft.

Chapter 8

EFFECT OF LIMITED DEPTH OF PERMEABLE STRATUM ON UPLIFT PRESSURES

8.1. In Khosla's theory, the uplift pressures are calculated on the assumption that the foundation medium is permeable and of infinite depth. It may so happen in the trough region of the river, beds of silt and clay may be existing between layers of sand resulting in stratification. Exact theoretical solutions for determining uplift pressures for stratified foundations are not available.

8.2. The effect of uplift pressures on limited number of distinct strata was theoretically studied by Pavlovsky.[10] In his study, he considers the following parameters:

i) Width of the apron 2b;

ii) Depth of the permeable strata T;

iii) Depth of sheet pile S.

To investigate the individual influences of the various relations Pavlovsky considers the following ratios

$n = T/b$

and $m = S/b$

Based on several computations, he came to the following conclusions:

i) The difference in pressures due to the effect of an impermeable sub-soil is less than 1% when n 5 and m 0.1;

ii) It is less than 10% if n 3 and m 0.4.

This determines the error consequent upon disregarding the impermeable sub-soil in calculations and experiments. In all his analysis, Pavlovsky assumes the sheet pile in the middle of apron.

8.3. The effect of stratification was also studied by Luthra[31 & 32] and Bansal[39] by electrical analogy methods.

Luthra carried out a series of experiments with two and three sheet piles. For two sheet piles at either ends of the floor with a model of a floor length l, the ratio of the depth upto the impervious layer, to the depth of the sheet pile (n) was kept equal to 1.0, 1.1, 1.25, 1.5, 2.0, 3, 4, 6, 8, 10, 12 and 15. The ratio of the depth of the sheet pile, to the length of the floor (m), was taken as 0.10 in all cases.

These results showed that there occurs an increase in pressure along both faces of the upstream sheet pile and a decrease all along both faces of the downstream sheet pile. As regards the floor itself the change in pressure at important points like $\frac{1}{4}$ l, $\frac{1}{2}$ l, $\frac{3}{4}$ l and l due to the change in position of the layer showed a general increase in uplift on the first half and decrease in the second half for values of n from 15 to 2. For n below 2, the variation was not definite.

In case of three piles, S_1 at the upstream end, S_2 at the middle, and S_3 at the downstream, showed that the pressures go on increasing on both faces of S_1 and on upstream face of S_2 and the pressures on other face of S_2 and on both faces of S_3 go on decreasing with decrease in n, maximum decrease being about 10%. Pressures at the end of sheet pile S_1 show progressive increase (upto 7%), while at the ends of S_2 and S_3 there is decrease.

Actual experiments were conducted by Luthra on a model of Panjnad Weir to exemplify the extent of effect of a stratified foundation on design. The actual Panjnand Weir, however, is found on a permeable stratum with three pile lines. In this experiment Luthra varied the depth of permeable stratum from the bottom of the pile. The results of his experiments are given at table 8.1.

It can be seen from the table that there is an increase in uplift due to the presence of impermeable layer and this increase is maximum (11%) when the impervious layer is at the bottom of the pile. The pressures thereafter decrease on the downstream pile, there is a significant increase of uplift pressures when the impervious layer is upto the bottom of the pile. As the depth of the pervious layer is increased the pressures decrease and are less than Khosla's values. These experiments reveal that on the upstream there is generally an increase of uplift pressures due to the impervious layer and on the downstream there is a decrease, except when the pile is embedded into impervious stratum.

The latter aspect gives a significant solution. On the downstream side the bottom of the pile, if kept slightly above the impervious stratum, there is a decrease in pressures. Advantage can be taken of this in actual practice. On the upstream, the cutoff can be taken down upto the impervious stratum. An increase in pressures at the upstream is of no consequence, since a head of water will be acting on the same. Further, it was stated earlier that on the floor the uplift pressures increase upto the middle of the floor. All intermediate pile if provided may

Table 8.1 - Comparative table showing uplift pressures below Panjnad Weir with permeable foundations of Infinite Depth and Finite Depth

Pile line	Upstream		Intermediate pile			Downstream pile	
Conditions	ϕ_D	ϕ_C	ϕ_{E_1}	ϕ_{D_1}	ϕ_{C_1}	ϕ_{E_1}	ϕ_{D_2}
i) Khosla's method	77.7	72	67.4	61.4	56.4	30	23
ii) By electrical analogy with impervious layer at:							
7 m	89	82.5	77.25	69.5	61.5	37	20.5
7.6 m	87	80	74	65	57.25	29	15.5
10.65 m	86	78.5	72	64.5	56.0	27	16.0
14.0 m	84	77.0	71	64	56.25	26.5	17.0
21.0 m	83	76	70	63	56.0	28.0	19.0
35.0 m	80	73	68	61	54.75	29.0	20.5

confine these pressures to the upstream.

In the arrangement of such cutoffs, Schokltsh[4] gives the following remarks:

"If the upstream cutoff reaches down to the impervious stratum the percolation is checked completely; and the ground-water pressure under the weir depends practically on the tail-water level. The pool at the toe of the weir is usually quite deep, and the cutoff there, which serves primarily as a protection against undermining, must also extend considerable distance into the sub-soil; it not be water-tight, but must be dense enough to prevent the washing out of sub-soil particles. This is the best arrangement; only under exceptional conditions will it be advisable to depart from this arrangement."

The remarks, however, willnot apply in actual practice as the sheet pile cannot be made completely leakproof and thus observations of Luthra apply.

8.4. The experiments conducted by Luthra are in respect of limited depth of permeable stratum only. No experiments were carried out by him for the case when the top layer is less pervious than the bottom layer. In this case regard, Bansal's experiments are of significance.

If the top layer is more pervious than the bottom layer, Bansal's experiments are in agreement with Luthra's. But when a stratum of lower permeability overlies a stratum of higher permeability, the results are reversed. Higher pressures are obtained on the downstream portion of the floor and lower on the upstreamuportion. For two layers of equal thickness and per-

-meability ratio 1/8, Bansal finds a maximum increase of 4% at the downstream end in comparison to the ideal homogeneous case.

8.5. If such a contingency occurs in any structure it may lead to serious pressure building on the downstream floor, which would be consequently endangered. Recourse has to be then taken to elaborate pressure-relief arrangements under the floor. The provision of this should not, however, be taken in reducing the floor thickness.

8.6. Diversion Structures on rock foundations

A good rock surface if available at a short distance below the bed of the river would form an ideal foundation for a diversion structure. In such cases the structure can be made narrower since higher intensities can safely be allowed. The long pavement, the protection works, the filter &c. can be safely omitted thereby achieving an overall economy in the structure.

Chapter 9

G U I D E B A N K S

9.1. Guide banks are, as the name implies, artificial embankments meant for guiding the river flow past a diversion structure, without causing damage to the structure and its approaches. The types of guide banks, their functions etc. have been outlined at Chapter 2.

In designing guide banks, their shape in plan, length of shanks, upstream and downstream of the barrage or weir, their heads, cross-sections, aprons and materials of construction - have all to be carefully considered.(2)

There are no rigid rules set for the layout of guide banks. The dimensions given by most of the investigators, are based on experience gained from existing works and the various relationships which are empirical and may suit only a particular local condition or may not. The adequacy or otherwise, of the dimensions of guide bunds should, therefore, in all cases, be verified by model experiments to avoid serious trouble during operation.

9.2. Form in plan

Whether a guide bank should be diverging, converging, or parallel, willbe governed by local conditions, so as to avoid aprons in deep water which, besides being extremely costly, cannot be properly laid.

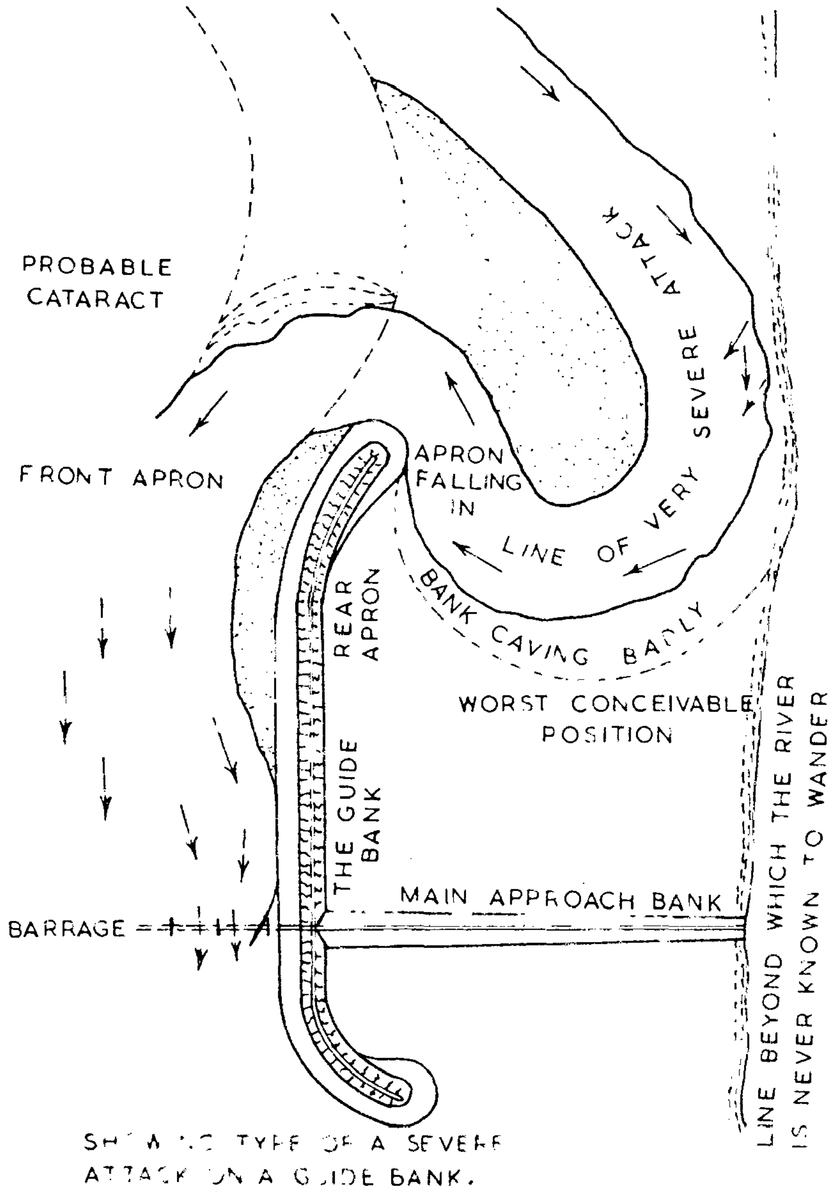
When a river is likely to meander on both sides upstream of the diversion structure, it is essential to make the guide banks symmetrical in order to straighten the current, under all possible conditions, and thus ensure uniform discharge distribution and even scour at the structure. The guide banks are also required to be aligned to create a favourable curvature for effective sand exclusion from the canals.

9.3. Length of the upstream and downstream parts

There is no hard-and-fast rule regarding adoption of the length of the guide banks, Views of different authorities are cited below.

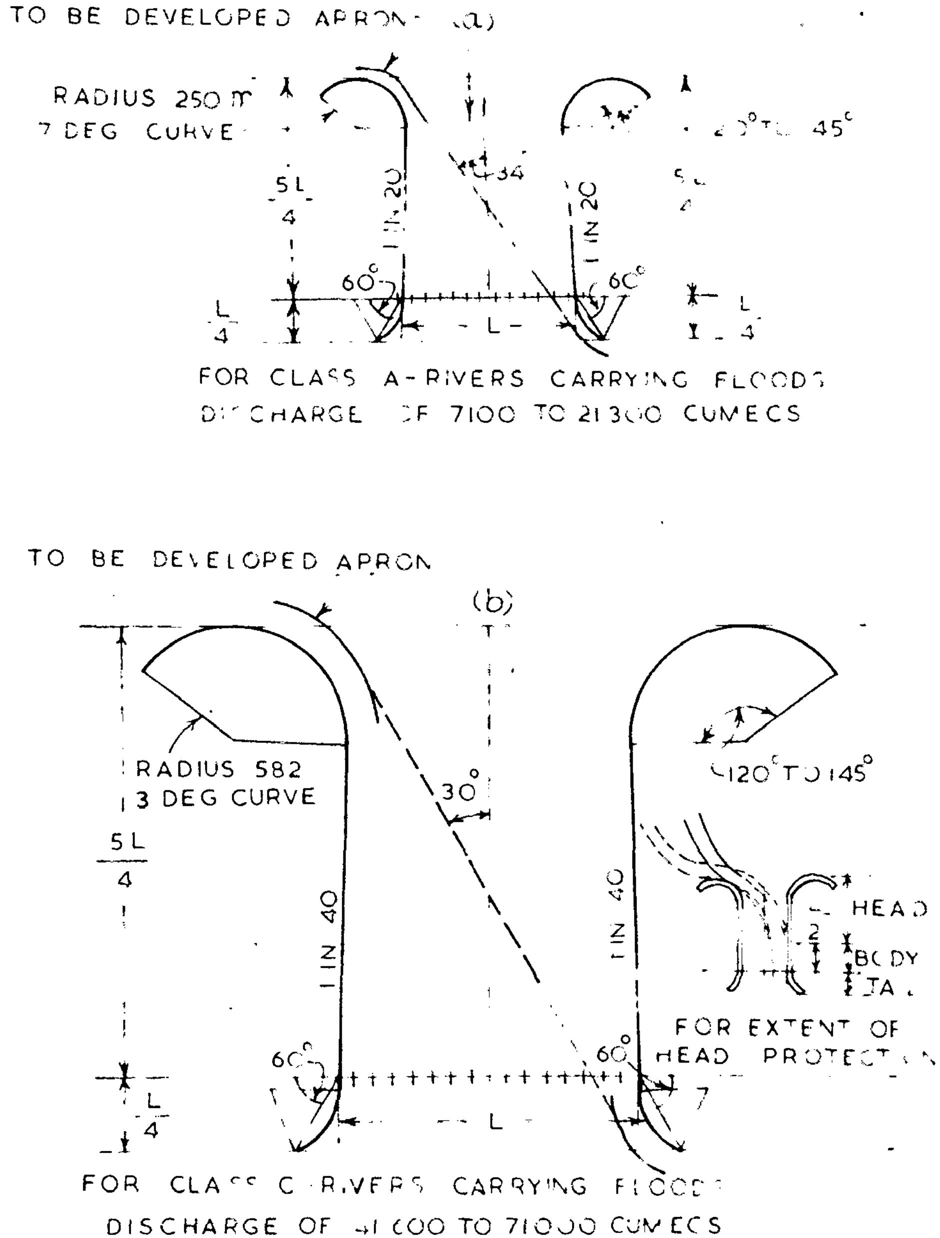
According to Spring, the length of the upstream part of a guide bank should be equalto or 10% longer than the length of the water-way between abutments (L); or even longer, if required to obviate the possibility of the river curving at the back and cutting into the approach bank. In designing, however, the worst bend prior to the development of a cutoff was to be considered (Fig. 9.1). The length of the guide banks on the downstream were suggested between 1/10 and 1/5 that of the water-way.

Gales's recommendations for the length of a training bank are based on the assumption that the structure is constructed well within the 'khadir' and is provided with two training bunds. He has recommended various lengths depending on the river discharges. For river with a discharge between 7,100 cumes to 21,300 cumecs, guide banks of upstream lengths $1\frac{1}{4}$ L with a convergence of 1 in 20 were considered by him to provide sufficient margin of safety and limit the angle of attack to less than 34° (Fig. 9.2 a).



PROBABLE CATARACT
FRONT APRON
APRON FALLING IN
LINE OF VERY SEVERE ATTACK
BANK CAVING BADLY
REAR APRON
WORST CONCEIVABLE POSITION
THE GUIDE BANK
MAIN APPROACH BANK
BARRAGE
LINE BEYOND WHICH THE RIVER IS NEVER KNOWN TO WANDER

FIG. 9.1



TO BE DEVELOPED APRON (a)
RADIUS 250 m
7 DEG CURVE
5L/4
1 IN 20
60°
L/4
L
FOR CLASS A-RIVERS CARRYING FLOODS
DISCHARGE OF 7100 TO 21300 CUMECS
TO BE DEVELOPED APRON
(b)
RADIUS 582
3 DEG CURVE
30°
120° TO 145°
1 IN 40
HEAD
BODY
FOR EXTENT OF HEAD PROTECTION
FOR CLASS C-RIVERS CARRYING FLOODS
DISCHARGE OF 41600 TO 71000 CUMECS

FIG. 9.2

For discharges between 41,600 cumecs and 71,000 cumecs, it was thought necessary to increase the upstream lengths of the guide bank to $1\frac{1}{2}$ L, to limit the obliquity of flow through the structure to 30° (Fig.9.2b). For intermediate discharges, adoption of the length of $1\frac{1}{4}$ L to $1\frac{1}{2}$ L was advocated.

Downstream length will have to be determined so that the swirls and turbulence likely to be caused by fanning outof the flow below the guide banks, do not endanger the structure. Guide banks on the downstream side were suggested to be 1/4th the waterway for all sizes of rivers.

The recommendations of Spring and Gales for the lengths of guide banks cannot be universally applicable and will need review in the light of subsequent experiences. One of the considerations in fixing the length of the guide bund is ensuring the safety of the approach bank. Every river has its own peculiarities, depending on its size, load characteristics, the terrain through which it flows, the nature of its banks etc. Hence, each should be considered individually.

9.4. Radius of head

If the ends of the guide banks are left square, due to the obstruction of flow, deep scours will occur as a result of formation of swirls as shown in Fig. 9.3a. To eliminate their occurrence the ends should be curved in plan. If, as shown in Fig.9.3.b, the guide banks are carried right upto the banks of the river, the cost involved nullfies the saving reckoned for narrowing the structure. The usual practice is to provide a curved head as shown in Fig. 9.3.c The angle to which it is curved being in the range of 120° - 145°.

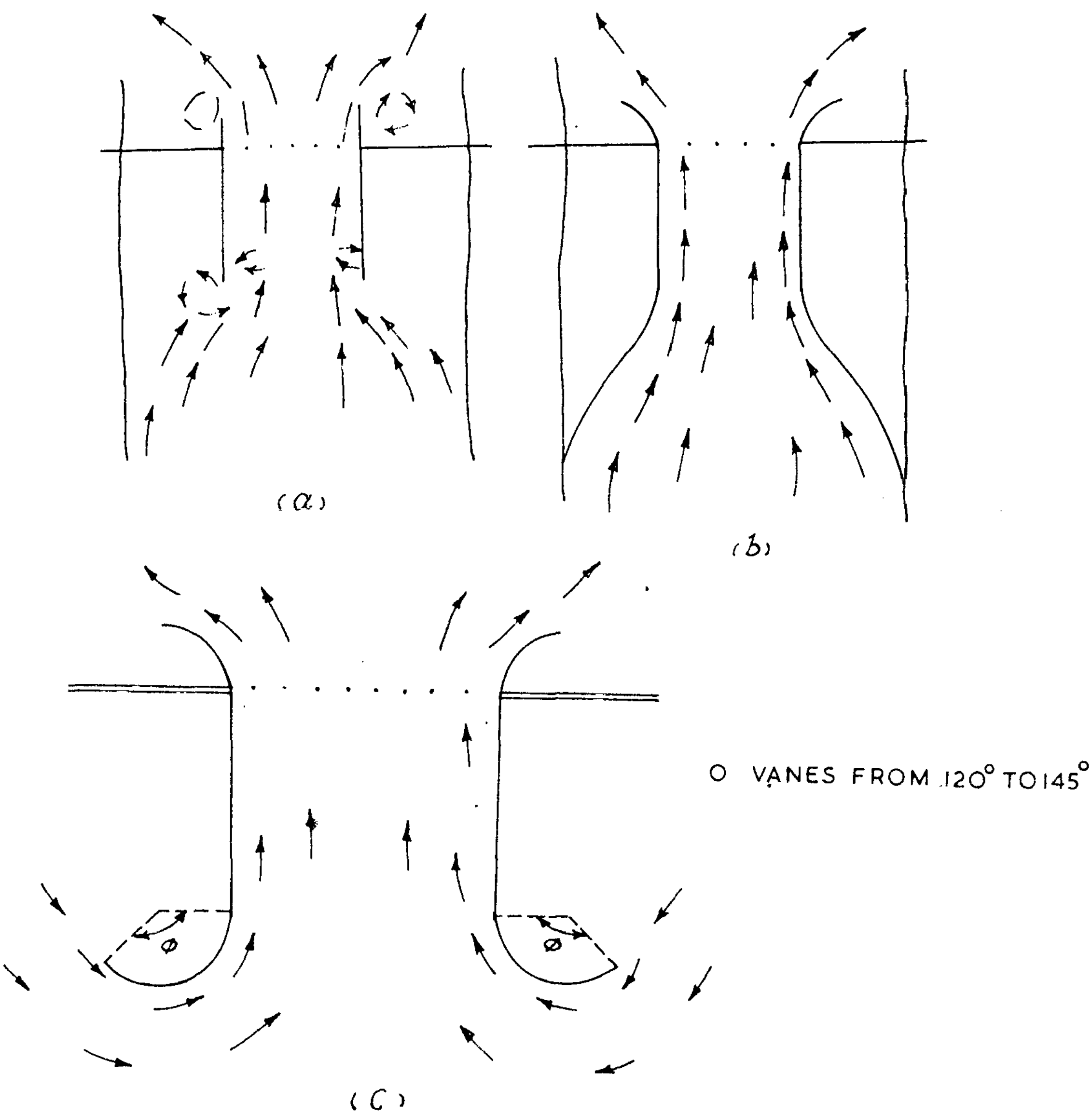


FIG. 9.3

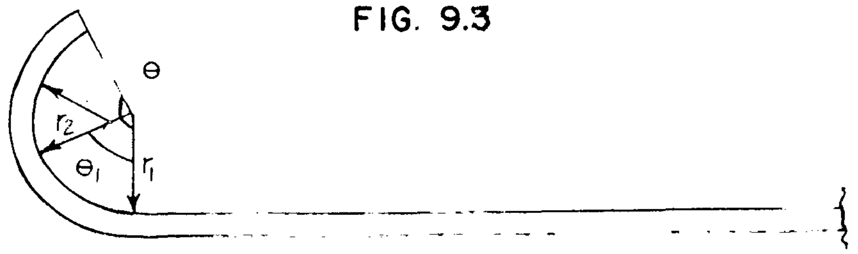


FIG.9.4 CURVES FOR GUIDE BANK HEADS

The radius that can be adopted depends upon the velocity with which the water moves in the river. According to Spring, water with a velocity of 2.44 m/sec. to 3.05 m/sec., which usually occurs, can easily fllow a curve of 183 m to 244 m. Also the radius adopted should further conform to the requirement for easy movement of the rolling stock for carriage of material during construction.

Gales recommended a 7°-curve (radius 249 m) for discharge between 7,100 cumecs to 21,300 cumecs and a 3°-curve (radius 582 m) for discharges in the range of 41,600 to 71,000 cumecs. For intermediate discharges, the raius to be adopted can be interpolated.

The values of radius of curvature, as recommended by Spring depending upon the bed material, probable maximum scour and the fall of the river, are summarised in table 9.1.

Considerable economy, consistent with smoother flow conditions at the head, may be achieved by adopting a composite curve of two or three different radii instead of single large radius. In this connection experiments by C. R. Rao[34], both by hydraulic models and electrical analogy have reference. His experiments consisted of: (i) Simple curvesfor the head of guide bank are used. Keeping a constant value for the radius (γ_1) of the curve, the angle θ to which the curve head is swung back, varied from 90° to 135°. The usual value varied from 120° to 145°. The He concludes that when simple curves are adopted an angle nearabout 100° for the head of the guide bank seems to be optimum value.

(2) In the second experiment the effect of composite curves

Table 9.1 - Radius of Heads of guide banks

Sand Classification	Probable maximum abnormal scour (metres)	Fall per km of river in cms.				
		4.5	9	14	19	28
		Radius of upstream curved end of guide bank (m)				
Very coarse	Under 6	60	75	90	105	120
	Above 6	75	90	110	130	150
Coarse	Under 9	90	110	125	150	200
	Above 9	105	130	155	175	210
Medium	Under 12	120	125	165	185	255
	Above 12	135	165	195	225	255
Fine	Under 15	150	175	200	230	255
	Above 15	180	215	245	275	305
Very fine	Under 18	180	210	240	270	300
	Above 18	240	270	300	330	360

Radius of curve of downstream ends of Guide bank	Half the above, with for minimum radius, that on which maintenance trains can be shunted.

on the pressure distribution was studied. The composite curve was composed of a simple curve in the beginning upto an angle (θ_1) and followed by another simple curve of a smaller radius (Fig. 9.4). The total angle adopted for the head was 135°. The radius of the latter simple curve was taken 1/2, 2/3rds, and 3/4ths of the radius of the preceding simple curve. The angle θ_1 tested were 90°, 60° and 30°.

From the above Mr. Rao concluded that (i) composite curves with the radius of the latter curve equal to half the preceding curve and starting at an angle θ_1 of 60° were found to be causing lower negative pressure and hence preferable over simple curves; (ii) Maximum velocity occurred very near the junction of the curved head with straight portion.

9.5. Shanks

The water level at the rear of the guide banks will be higher than that on the river side, due to heading of water, by absorption of the velocity head, as the area between the guide bank and the river bank serves as a still water area and soon silts up. Therefore, adequate free-board has to be provided with respect to the water level at the rear of the guide bank instead of the maximum water level in the river.

9.6. Slope protection for shanks

For a core consisting of sand, the material should be obtained from the river side, and not from the rear. To prevent the sand from being washed away by the current, the face should be pitched suitably at slopes not steeper than 2:1, carried down to the low water level. The upstream face of the guide bank has to be armoured with suitable pitching to protect it from every conceivable kind of attack. Stone weighing 35 to 55 kgs are generally used and these are not moved even at velocities of 5.5 m/sec. (This has been amply borne out by experience.) Concrete blocks are equally good; round and smooth boulders though are used where available locally, do not have the advantage of natural interlocking as in the case of angular stones and the latter should therefore be preferred. The thickness of pitching to be provided is dependent upon grade of the river and size of bed material.

Thickness of pitching on the slope, according to Spring, is given in the table 9.2.

Table 9.2.

Fall per km of river in cms	4.5	14	19	28	38
Sand classification	Thickness on slope in cms				
Very coarse	40	48	56	64	71
Coarse	56	64	71	79	86
Medium	71	79	86	94	101
Fine	86	96	101	109	117
Very fine	101	109	117	124	132

Hand packing and careful gradation of the stone, with smaller stuff, such as quarry refuse or even burnt bricks, between the sand and the large stones are necessary to prevent the sand being sucked out by high velocity flow. By means of this, the thickness may perhaps be safely reduced by 15 to 22 cms all round. Because of the extra severity of attacks on the upstream head of the guide bank, pitching should be made 25% thicker here than elsewhere. Pitching dropped through deep water or which will launch automatically, may not be uniform; consequently, about 25% more stone should be used when dumped.

Gales recommends the following thickness of pitching for the head, body and tail depending upon the discharge vide table 9.3.

Inglis, however, is of the opinion that discharge is the proper criterion for determining the thickness of stone pitching. He considers that slope and grade of the material are dependent variables. He gives the following formula, taking the available data into account:

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

where T is the thickness in metres and Q, the discharge in cumecs.

It is obvious that the degree of protection depends on the state of turbulence. As turbulence increases with the grade of material and coarse material exposed on the river bed, larger should be the stone protection. For example, stone which would form a suitable protection if the ned material were fine sand, would be inadequate and be washed away, if the bedand banks were composed of shingle and boulders. In boulder rivers, protection generally takes the form of large concrete blocks or large wire crates containing boulders.

For a river with high discharge in the plains, the values given by Spring will be the lowest; Inglis's,the higher; and

Table 9.13.-Thickness of stone pitching on the slope (Gales)

Rivers	Rivers with 7100-21 300 cumecs		Rivers with 21 300-42600 cumecs		Rivers with 42600-71000 cumecs		Remarks
	Head	Body & Tail	Head	Body & Tail	Head	Body & Tail	
Pitching stone thickness soling ballast (cms)	105	105	105	105	105	105	To be hand-set ballast
Total thickness of stone covering (cms)	124	124	127	127	130	136	Broken to pass 6-cm ring

Gales's would be the highest. It is for the designer to take the suitable value given by Gales or Inglis.

9.7. Apron

The face of the guide bank is protected upto the river-bed level by pitching, so that during floods, the sloping face is not damaged. Scour, however, would occur at the toe with consequent undermining and collapse of the pitching. To obviate such damage to the slopes, a cover, known as apron, is laid beyond the toe on the horizontal river bed, so that scour undermines first the apron, starting at its farthest end and works backwards towards the slope. The apron,in case of bed scour, will launch to cover the face of the scour, with stone forming a continuous carpet below the permanent slopes of the guide bank. Adequate quantity of stone for the apron has to be provided to ensure complete protection of the whole of the scoured face. The quantity will obviously depend on the apron thickness, depth of scour and the slope of the launched apron. These are considered below:

i) Estimation of scour depth and thickness of launched apron:

Due to the constriction and rigidity of the structure, more scour occurs than in a straight reach of an un-obstructed river. Spring recommends that the guide banks should be designed for the worst abnormal scour to be found in the river.

Gales's method of arriving at the probable scour depth is given in table 9.4.

The thickness of the launched apron has been variously proposed by different authorities. Spring suggests a thickness of

Table 9.4 - Gales's method of arriving at deepest scour to be adopted in design

	7,100 - 21,300 cumecs	21,300 - 42,600 cumecs	42,600 - 71,000 cumecs
Observed deepest scour below L.W.L., along a soft cutting bank in the bend at 3/4 falling flood	x_1	x_2	x_3
Add 33% 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 and for severe attack on the guide bank head -			
For body and tail of the bank	25%	32%	45%
For head of guide bank	50%	63%	90%
Deepest scour to be adopted, below L.W.L. -			
For body and tail of the bank	$1.66x_1$	$1.75x_2$	$1.93x_3$
For head of the bank	$2.0x_1$	$2.17x_2$	$2.53x_3$

1.25 times the slope thickness for the apron as minimum, as the apron will not form a uniform carpet in launching. For the rivers liable to deep scour, it may be increased to 1.5 times. He recommends further that the thickness of apron, at the junction of apron and slope laid, should be the same as that of the slope stone, but should be increased in the shape of a wedge towards the river end, where severity of attack and, hence, possibility of loss of stone is greater.

Gales recommends a uniform apron with a berm at the toe (Fig. 9.5) of the slope. These also vary for the same discharge ranges as for slope thickness (Table 9.4). These are empirically arrived at by Gales, based on his experience at the Hardinge Bridge over Ganga at Sara. Since these are based on experience on one river and therefore are only a rough guide.

Incindetally, these are more in vogue than Spring's recommendation.

Khosla has suggested the following values for the design offlexible loose stone aprons of guide banks in terms of Lacey's scour depth D, where

$$D = 0.47 \left| \frac{Q}{f} \right|^{1/3}$$

in which D = scour depth below high flood level (m);

f = silt factor = $1.76 \sqrt{d_m}$

d_m = grain size diameter in mm.

.../

Location	Range of scour depth	Mean depth to be adopted
Noseof guide bank	2.D to 2.5 D	2.5D
Transition from rock to straight portion	1.25 D to 1.75 D	1.5D
Straight reaches of guide bank	D to 1.5 D	1.25D

Experiments conducted at the Poona Research Station with available data for existing guide banks showed that for the portion of the shank in the vicinity of the piers, the maximum scour would be $2D_{Lacey}$. For large radius of the guide banks the scour depth was of the order of $2.75D_{Lacey}$. These figures can, therefore, be adopted.

ii) Slope of the launched apron:

Spring and Gales have suggested a slope of 2:1.

Model observations have shown that an apron does not launch satisfactorily unless the angle of repose of the underlying material is flatter than that of the protective work. In the model, the under water stable slope varied from 1.57:1 to to 2.35:1. With one man stones, the angle was about 25:1. The flattest angle was obtained with rounded stone laid on Ganga sand of diameter 0.29 mm. The angle depends to a small extent, upon the velocity, giving a flatter slope for higher velocity and stronger attack. With angular stone, the slope is steeper than rounded stones.

Observations at guide banks on various rivers have shown that the actual slopes of launched aprons range from 1.5:1 to 3:1, and are even flatter. The average is 2:1. The face of launched apron should not, therefore, be assumed steeper than 2:1 nor flatter than 3:1.

Bell, the originator of the guide bank system, recommended a breadth of apron equal to 1.25 times the scour depth below the bed level at which the apron is laid. Spring and Gales suggest that this should be less than 1.5 D for shank and 2D for the head. In addition to 1.5D Gales provides an extra 4.5-m to 7.5-m width of apron at the foot of the slope, which he calls berm, intended to ensure permanence of the pitching stone on the slopes. Experience and experiments have shown that where scour was gradual, the slope and quantity of stone were practically the same, whether the apron was laid deep and narrow or shallow and wide; but where scour occurred rapidly a shallow wide apron would launch more gradually and evenly, than a deep and narrow one. Hence for any normal case 1.5 D appears to be allright.

iii) Stone quantity in apron:

Spring advocates a thickness equal to that of the shank slope pitching, at the inner end, and increasing upto 2.76 times at the outer edge. The total stone quantity then will be (vide Fig. 9.5) 2.82 DT and that the apron is assumed to launch to its final shape of 2:1 when full scour developes. The slope length then will be $\sqrt{5}$D. With an average thickness 1.25T the quantity of apron stone necessary is $1.25T \times \sqrt{5}D = 2.80$ TD per metre width. This distribution, however, is not adequate considering the quantity of stone as can be seen. In the first half, the apron stone as provided shall be

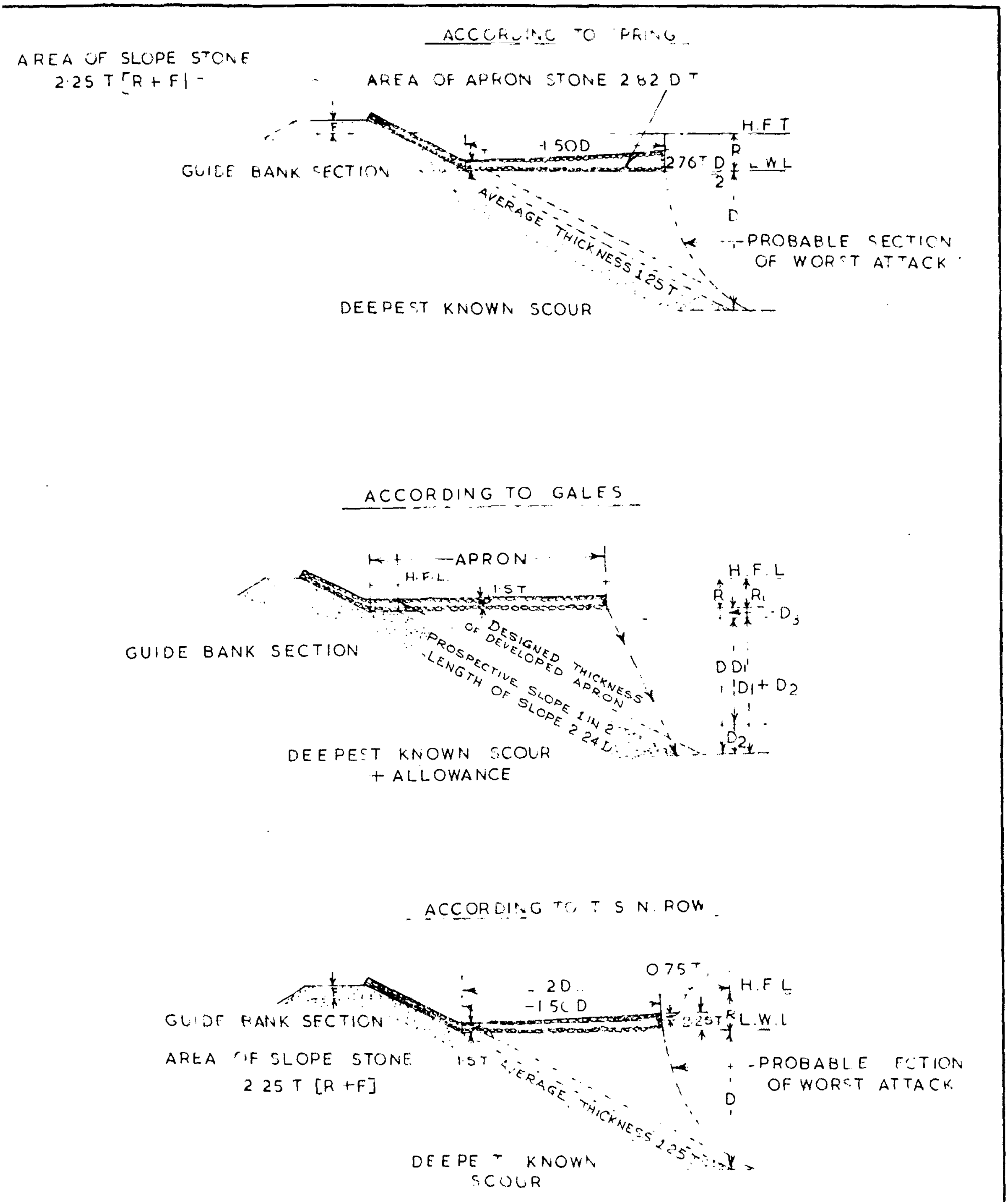


FIG. 9.5 SHOWING DESIGNS OF GUIDE BANK SLOPE AND APRON PROTECTION PROPOSED BY SPRING, GALES AND T.S.N. ROW.

$$\frac{T + 1.88T}{2} \times \frac{1.5D}{2} = 1.08TD$$

Actual requirement will be

$$5 \times \frac{D}{2} \times 1.25T = 1.4\ TD$$

Thus there is a deficiency of stone.

This has been tackled differently by different authors. Gales advocated provision of a berm in addition to length of the apron, which appears to be superfluous with apron designed with due care and hence need not be adopted.

While discussing the distribution of stone in the apron, T. S. N. Rao suggests in his 'History of Hardinge Bridge' a modification of Spring's design. Dimensions of the laid apron according to Rao are also shown in Fig. 9.5 He assumes that over and above the normal thickness of launched apron, T, additional stone to be provided for irregularity in launching, should vary in proportion to the depth to which each section of apron is required to launch. For this, additional stone, sufficient to form a thickness of 0.25T on the developed slope, should be provided and laid with a uniformly increasing thickness from the inner to the outer edge of the entire apron. The area of this additional apron will be $0.25T \times \sqrt{5}D = 0.57\ TD$, spread over a width of 1.5D. Hence the average additional thickness becomes 0.375T, while the thickness of the additional triangular stone wedge at the outer edge is 0.75 T. This method gives a fair distribution.

Whatever be the type of apron section, a certain dispersion, which will be maximum at the outer edge, is unavoidable when the apron is launched. As a general practice, an adequate thickness of apron should be provided at the toe of the slope to ensure a strength after launching, equal to that of the stone pitching on the slope face. Additional stone, out of the total apron quantity worked out earlier, should then be provided for irregularity of launching and washing away of stone; this can be better distributed in the apron in triangular wedge shape with maximum thickness at the outer edge as suggested by Rao.

Experiments at the Poona Research Station have shown that for satisfactory launching, bed material should scour easily and evenly. With an apron laid on the river bed consisting of alternate layers of sand clay, stones slide down as sand layers scour and clay layers subside, causing un-even cliffs, so that the apron cannot launch uniformly. Stones fall to the bottom and are washed away. Clay banks cannot therefore be used as a dependable foundation for aprons. Conditions occur, however, where clay beds are unavoidable. Heavy maintenance has then to be done by keeping a sufficient reserve of stone at hand, to fill the un-even scour holes and form a uniform slope of stone.

9.8. Free boards

It is necessary for any hydraulic structure that some free board over the maximum anticipated water level, should be provided. However, no hard-abd-fast rule exists for the provision of such free boards. In many structures the free board is provided arbitrarily based on the discharge; the higher the discharge, higher will be the free board.

In the Uttar Pradesh, the free board is provided for a flood discharge of 1 in 500 years. The Central Water & Power Commission provide a free board of about 1m and check its adequacy for a super flood equal to design flood + 25%.

A higher free board wouldmean uneconomical and a low free board may lead to risk. It is, therefore, necessary that this aspect should be carefully studied while providing in any structure.

9.9. Specific examples

It has already been stated at Para 2, that the layout of a guide bund is mostly governed by local conditions. In this connection the alignments of the guide banks for the Jumna Weir in Assam, where the bunds on either side flare out to make different angles with the normal to the weir axis - are a good example.[43]

To meet the irrigation demands of the drought-affected areas in the district of Nowgong (Assam), it is proposed to construct a weir about 304.80 m long on river Jumna and to draw off a discharge of 16.99 m^3/sec. into the canal. Model experiments were conducted at the Central Water & Power Research Station, Poona, for finalising the layout. The design discharge of the weir is 3,143.16 m^3/sec.

Experiments under existing conditions, i.e. without the weir, showed that the flow was normal to the proposed weir alignment (i.e. design) only upto the bankful stage, viz. 226.53 m^3/sec. and with the increased discharge, the flow was seen to be oblique, the obliquity increasing to about 30° for the design discharge of

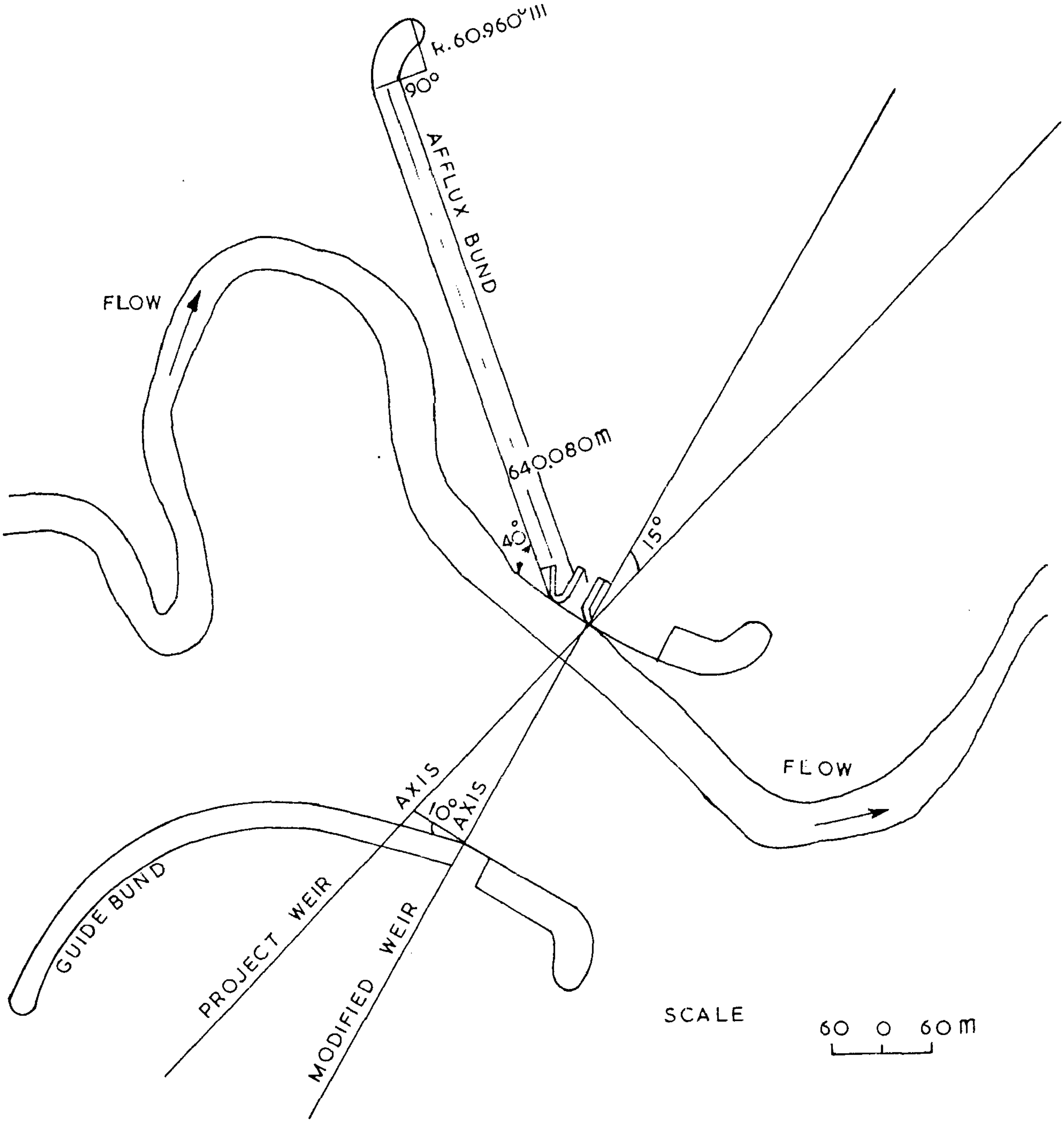


FIG. 9.6 ALIGNMENT OF GUIDE BANKS FOR JUMNA WEIR (ASSAM)

3,143.16 m^3/sec. The alignment of the weir was accordingly tilted by about 15°, so as to be normal to the flow at a stage of 707.92 m^3/sec. and above (normal discharge). The position of the left abutment, however, remained unchanged (Fig. 9.6). Looking to the right bank spills, it was also found necessary to flare the right upstream guide bund by 10° to the normal to the weir axis, so as to make the spill-flow hug the guide bund. No change was effected in the left guide bund at this stage. At discharge 707.92 m^3/sec. it was observed that the flow line was normal, but a very heavy attack persisted on the left guide-cum-afflux bund due to the sharp curvature of flow. It was, therefore, considered desirable to retire the left guide-cum-afflux bund to ensure safety. Accordingly, it was flared at 40° to the normal to the modified weir alignment and curve of radius 60.96 m was given to the upstream end.

Incidentally, the guide banks are located on clay beds. Hence the aprons would not launch properly and some maintenance would be necessary.

Chapter 10

C O N C L U S I O N S

10.1. The location of any diversion structure should aim at a trouble-free operation consitent with economy. The various factors involved in this are the location of the diversion structure, fixation of its water-way, layout and hydraulic design of the various components &c. Much work has been done in this field, but still the laws of river flow and its consequent effect on a diversion structure are yet not fully established. A wide gap exists between the known principles and their application.

For design of any hydraulic structure it is necessary to have knowledge of the application of the various principles involved in design and their limitations. These are therefore briefly described below.

10.2. Proper location of the diversion work and the intake are of utmost importance for a trouble-free performance. The alignment of a diversion work should be, as far as as practicable, normal to the river flow at all stages. In view of the prevailing basic hydraulic and sediment principles, some of which have already been discussed, it would be most practical to locate intakes on the outside of river bends. In some locations there may not be a suitable river bend near the site; or a site on a bend may be unfavourable by reasons of the limiting geologic features. In selecting the site under these circumstances, it is necessary to compare the advantages of several sites, taking into account the geologic, hydraulic, structural and economic considerations. A final selection of the site can be made, when the best combination of contingent factors is attained with respect to operation of the diversion system.

10.3. Next comes in the line from the consideration of performance and economy for a diversion work is the fixationof a suitable water-way.

The old barrages in Punjab are provided with a wide waterway with the result that sand shoals tended to form upstreamof them. Ras Weir on the Jhelum, Panjnad Barrage on the Sutlej and Khanki on the Chenab, are some of the typical examples of old weirs with wide water-way. Comparative figures of the waterways of some of the existing structures, with Lacey's water-way (which is used as a standard for comparison) shows that the looseness factor was as much as 1.98 in the case of Merala Weir on Chenab (constructed in 1907).

It needs to be, however, emphasised that certain considerations are of vital importance in adopting the criterion of looseness factor based on Lacey's formula in fixing the width of a diversion structure.

One such consideration is the discharge, for which the looseness factor has been worked out. Generally, the design flood discharge is adopted in different projects according to different criteria. What is really important in designing the weir or barrage water-way for ensuring free approach, is not the design flood discharge of a very low frequency, as would generally be the case, but normal flood discharge which has got high frequency. The structure should, however, be able to pass the maximum design discharge with requisite afflux. The Dakpathar Barrage is a typi-

-cal example of such case.

Question may arise, how far is it correct to apply looseness factor in the design of barrages and weirs. Lacey,[42] in one of his discussions on the subject, stated "the width ($P_w = 4.75\sqrt{Q}$) applied to an unrestricted river which was free to scour and had well-defined thalweg. When it was applied to the design of an irrigation diversion-work constructed across a river much depended on whether the work was a weir, in which case there was a horizontal bar across the channel, or the work was a well designed barrage. In the latter case, it was assumed that with all gates fully open the river would flow unimpeded through the barrage and would not differ greatly from a bridge."

Lacey's statement may be taken to suggest that looseness factor based on his formula may be adopted as a design criterion for barrage water-way and not for weirs. It, however, needs to be realised that Lacey's width formula though of the correct form and with correct index has been found to require different constants to fit data of different rivers and canals. Inglis had found that statistical analysis of the observations on the Lower Chenab Canal, presumably belonging to the regime type, yielded a wide range of divergence from Lacey's values. According to him the constant in P-Q formula varied from 0.8 to 1.45 times Lacey's value. Thus, even in case of canals in regime, Lacey's factor of 4.75 is not strictly constant. The divergence in case of rivers is known to be quite considerable. The rivers carrying heavy loads of sediment are known to be wider and shallower giving a higher value of constant. Lacey suggests a value of 1.5 for the looseness factor for rivers carrying heavy sediment load and 1.25 for others.

The abovediscussion leads to the inference that the present design practice in respect of determination of water-ways for barrage in terms of looseness factor are not sufficiently rigorous for ensuring prevention of sand deposits upstream of the barrage.

The Narora Barrage, the Mundali Weir, the Salandi Barrage, are typical examples of diversion works where the Lacey's formula was not the governing criterion. The popular belief that the cost of training and protection works would increase if a narrow waterway is provided, due to increase in intensity, should always be weighed with the economy achieved in reduction of a couple of bays.

10.4. For a trouble-free performance of anyproject, sediment control is an essential requirement and it has to be done systematically. A suitable design of barrage or weir, undersluices, divide wall, regulator , guide banks etc. goes a long way to minimise sediment entry into the canal.

A divide wall extending two-third times the distance of the upstream end of the canal regulator from the barrage is supposed to give best results. This, however, cannot be taken rigidly. In the case of Mundali Weir, the Kemri Barrage, the divide wall extends the full width of the regulator. In Trisuli Barrage, the divide wall extends covering $1\frac{1}{2}$ bays only. No divide wall was provided in the Nangal and Salandi Barrages.

Dominy[41] while discussing the various types of sediment exclusion devices adopted by the U.S.B.R. states that curved divide walls have been found to be efficient in excluding sediment from canals. The divide wall extends in an upstream direction along a curved path. The direction of curvature is such that the wall forms an approach channel in which the canal inlet is on the outside of the curve. The radius of curvature and position of the divide wall are determined from model experiments.

10.5. The layout of the head-regulator in any diversion structure needs particular attention. Several angles have been advocated for the take-off. The Punjab practice advocates 10° to 12°, whereas in Uttar Pradesh as much as 20° - 21° have been provided. The European theory recommends a 30° offtake angle. According to Leliavsky, Schoklitsh, this is thebest angle. This was also substantiated by experiments at University of Roorkee. Such an angle, however, has not been provided in any barrage in this country, though in several barrages and weirs the canals take off at 90°.

The crest of the head-regulator should, in all cases, be kept higher than the crest of the undersluices for a trouble-free performance. This has been amply borne by experience and experiments. The trouble at Krishna anicut is an example of a recent work where the crest of the head-regulator is kept lower than the crest of the undersluices.

10.5. The depth of sheet piles on upstream and downstream of barrage is determined by Lacey's formula

$$\boxed{R = 1.35\left\{ \frac{q^2}{f} \right\}^{1/3}}$$

It is recommended that it will be sufficient if the sheet piles are taken to a depth equal to R below the H.F.L. But it was seen that in almost all the barrages, this provision has been exceeded. According to Leliavsky, the depth of the sheet pile below the floor should be equal to the head of water above the floor. This relation was established by actual observation of scour holes. Unfortunately, no such data is available for the barrages and weirs in this country.

10.7. The downstream sheet pile is, however, governed by the scour as well as safe exit gradient conditions. For a particular factor of safety of exit gradiant; deeper the pile depth, lesser shall be the floor length. But there will be an increase in the uplift pressures. Consequently, the floor thickness increases. It is therefore necessary that the influence of one factor over the other should be worked out for an economic design.

10.8. It is desirable that the length and level of the cistern should be determined from model experiments. The level of the basin should, if possible, be raised by provision of appurtenant works.

10.9. It is customary to provide the balance floor length and the upstream after accounting for the floor length required for the cistern, the glacis and the crest. It is, however, not desirable to provide a lengthy upstream floor. This floor is always under a head of water and is never open for inspection. Should any cracks develop this may lead to serious trouble and piping may start by consequent reduction in floorlength.

10.10. Uplift pressures beneath the floor are determined by

Khosla's method, as this gives fairly accurate results and is quick. The limitation of this method is that it is not applicable to stratified foundations. In such cases, it is necessary to determine the magnitude of such uplift pressures by Electrical Analogy method. If the impervious layer is at a depth somewhat lower than the estimated scour depths, it may be worthwhile to tie the upstream sheet pile line to the impervious layer while the downstream sheet pile line may be left above the impervious layer for release of such uplift pressures as may build up by leakage through the upstream pile line. All sheet piles are leaky to a lesser or greater extent depending on the tightness of their joints and possibility of lea ing or damage during driving. Thus even after tying the upstream pile line to the impervious layer, seepage pressures may build up under the floor in course of time. Their magnitude is difficult to determine. In view of this, elaborate pressure arrangements should be made below the downstream floor, which consist of longitudinal and cross drains with perforated pipes. Such an arrangement has been provided in the case of the Nangal and Salandi Barrages.

10.11. The alignment, length and radius of the head etc. for the guide bunds should be best determined by model experiments. As already stated the recommendations by Spring or Gales for the length etc. are not universally applicable. The Jumna Weir in Assam is a typical example. It will be desirable to provide varying radii for the headof the guide banks as it would ensure economy. Much work has to be done in this regard to evolve some standard practice.

10.12/

10.12. Observation of uplift pressures is desirable for any diversion structure for which instrumentation should be provided for. The instrumentation is a MUST when pressure-relief arrangements are provided.

10.13. Difference of opinion exists in respect of the wearing coat for consideration in the total floor thickness. But the fact remains this wearing coat is primarily meant for wearing out and in actual practice the provision differs for different stages of the river in which the structure is located. Generally, in boulder stage stone sets are provided as in Tajewala or Bhimgoda Weirs and Gandak Barrage. In alluvial rivers the wearing coat consists of a rich mix of cement concrete with reinforcement. Whatever may be type, this should not be taken into account for considering the total floor thickness.

10.14. For any diversion structureit is necessary that the concrete mix to be used in the structure should be designed first based on the properties of the availability of materials, in order to assess its correct seight and strength in design. Considerable economy in the structure, as also safety on certain occasions, can be ensured.

10.15. The design of any diversion structure should also take into account construction aspect also. The construction is phased to extend a coupleof years, a few bays being taken up for construction every year. It is, therefore, necessary that a lineof sheet piles should be driven along the edge of the last bay taken up for execution, joining the upstream and downstream pile lines (parallel toriver flow). This would prevent scour beneath the floor and also facilitate continuity of work. The copper seal joint can be left embedded in the concrete at this end bay to connect up with the subsequent year's work.

In someof the barrages like Gandak and Kosi, double piers are provided at the junction of these two floors.

10.16. The above are a few examples of the principles adopted in some of the barrages and weirs already constructed or under construction. It is necessary tostudy the actual performance of some of the more recent works and compare the same with the assumptions made in design. This would go a long way to establish a more rational approach to the various problems in the design of diversion structures.

The actual studies that should be made in this regard are:

i) The water-way plays a vital role in any diversion structure, particularly in regard to shoal formation. The water-way is generally based on design flood. This may hold good for diversion works located below the storage dams but would not work for weirs to be provided on rivers where regulating storage reservoirs do not exist. In such cases, it should be based on predominant flood. Studies are called for to decide the predominant flood for which the water-way should be provided.

ii) It is an accepted principle that a river regime is disturbed after the construction of the diversion structure and that it re-establishes after a lapse of a few years. There is little data available at this stage on this point. Such data will be useful to verify the assumptions made in the design and should prove a useful guide to future works.

iii) In stratified foundations it is necessary to take the upstream sheet pile into the impermeable stratum. But most sheet piles would be leaky to a certain extent, resulting in building of pressures. The effect of provision of a positive cutoff with deep penetration into the permeable strata needs to be studied to see if an advantage can be taken to reduce floor thickness with such arrangement.

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