MANUAL ON BARRAGES AND WEIRS ON PERMEABLE FOUNDATION

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CENTRAL BOARD OF IRRIGATION AND POWER
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NOTE : The statements and opinions expressed by the authors are their own and not necessarily those of the Central Board of Irrigation and Power.

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FOREWORD

The Central Board of Irrigation and Power has been issuing a number of Manuals for the guidance of the practicing engineers in persuance of its mandate to pool the available technical knowledge and experience, and utilise the same for the further advancement of knowledge and realising economies in planning, design and construction of Irrigation and Power Projects. The recent Manuals on Canal Linings (Technical Report No. 14), Irrigation and Power Channels (Publication No.171) and Hydraulic Tunnels (Publication No.178) have been welcomed by the profession. In view of this response, the CBI&P has planned to prepare and issue many more manuals covering various aspects of Water Resources Development and Utilisation. The present Manual on Barrages and Weirs on Permeable Foundations is one of this series.

This manual is intended to serve as a guide for the practicing Engineers in the layout and design of barrages on the basis of examples of diversion structures already built and experience gained so far. The present Manual in two volumes covers both head works and appurtenant works including regulators, silt excluders, fishways, guide bunds, etc. Recent trends in the design of such structures have been given a good coverage. Relevant drawings of some of the important projects have been included in this Manual.

This Manual on Barrages and Weirs on Permeable Foundation has been prepared by the Central Water Commission. The CBIP is thankful to the officers and staff who have assisted in the preparation of this Manual. It is hoped that the Manual would be found to be useful by our designers and field engineers.

C V J Varma

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Member Secretary
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NEW DELHI

PREFACE

Irrigation has been in vogue in India for over a number of centuries. It was a common practice to construct temporary diversion works across rivers and divert the river flow through canals for irrigation. Subsequently, diversion works of a permanent nature were constructed. Thus, the grand anicut across river Cauvery is believed to have been constructed during the second century and the structure is still in service with improvements carried out a few times. This was truly a master-piece in engineering during those times.

The Moghul rulers also took keen interest in constructing canals for irrigation. However, these were built without any permanent headworks and there were many failures experienced. Again during the 19th century, diversion works across major rivers were constructed. There were two distinct types of construction. In one type, only dry stone rubble was used without any mortar. The Okhla weir across river Yamuna in Delhi, the Jobra and Birupa weirs in Orissa State are examples of this type. In the other type, stones with lime mortar were used for construction. The famous Krishna anicut and Godavari anicut built by Sir Arthur Cotton fall into this type.

During the early part of the 20th century, a number of barrages were built, particularly in Punjab across the Indus river system. The practice of design of barrage during this period was to adopt a formula similar to Bligh's theory for determining the overall dimensions of the weir. The difference of water-level between the upstream and downstream of the structure used to be small. A few of these structures collapsed either due to piping or due to excessive scour downstream on account of the hydraulic jump sweep-out. This led to the scientific investigation into the causes of the failure of structures on permeable foundation, particularly by Dr. A.N. Khosla. The Central Board of Irrigation and Power brought out a publication on the "Design of Weirs on Permeable Foundation" in 1936.

After India attained Independence in 1947, a good number of diversion structures have been constructed and are under construction. In order to serve as a guide for the planning, layout and design of barrages on the basis of examples of diversion structures built and experience gained so far, this "Manual on Barrages and Weirs on Permeable Foundation" has been prepared. The Manual includes both headworks and appurtenant works including head regulators, silt excluders, fishways, guide bunds, etc. This Manual is intended to serve only as a guide and to indicate the recent trends and factors used in the design of such structures. The relevant drawings of some of the important projects like Kosi Barrage, Gandak Barrage, Yamuna Barrage, Khanabad Barrage, Godavari Barrage, etc., are included in this Manual.

Any suggestions for the improvement of the Manual in the future editions of the same will be gratefully acknowledged.

The Manual on Barrages was prepared in the Barrage & Canals Designs Directorate-I of the Central Water Commission by Shri N. Suryanarayanan, formerly Deputy Director.

The efforts of the officers and staff who have assisted in the preparation of the manual are greatly appreciated and my sincere thanks to them for bringing out this long awaited "Manual on Barrages and Weirs on Permeable Foundation."

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CHAPTER 1

Introduction

1.0. Diversion of the river waters is achieved by the construction of barrage or weirs across the river. The diversion of water may be required for purposes of irrigation, hydro electric power, drinking water supply or industrial water supply including cooling waters for thermal power stations, navigation, inter-basin transfer of water or combination of any of these.

1.1. The objects of constructing a barrage or weir for the required purpose are :

(*i*) The water level in the canal is required to be at a calculated level so that it may be able to command by flow a particular area to be irrigated or for feeding the power station, etc.

(*ii*) The full supply level of water required for this purpose can be obtained only if the river is tapped at a considerable distance upstream of the command area. To reduce the idle length of canal before it starts serving the area, the water can be tapped at a lower point subject to suitability of site by construction of a barrage or weir and having a suitable pond level.

(*iii*) The fluctuation in the supply level in the river can be reduced.

(*iv*) Sometimes it may be necessary to store small quantity of water for tiding over small periods of short supplies For this, storage weir or barrage with small storage can be had.

(*v*) To have better control over silt movement and to channelise the river to a permanent course, diversion structures can be provided.

1.2. Hence, a weir may be defined as an ungated barrier across a stream to raise the water level in the stream. Similarly, a barrage may be defined as a barrier provided with a series of gates across the river to regulate the water surface level and pattern of flow upstream. It is distinguished from a weir in that it is gated over its entire length and may or may not have a raised sill.

1.3. The diversion structures are located in the following four regions viz., (*i*) the mountainous or torrential (*ii*) the sub-mountainous (*iii*) the trough and (*iv*) the alluvial and deltaic. Locating the structure in any of these regions has its own advantages and disadvantages. These are discussed below.

1.3.1. The torrential of mountainous region is suitable for locating a diversion structure required for hydel power schemes due to the availability of high heads and less siltation problems. However, so far as structures required for irrigation schemes are concerned, the mountainous and sub-mountainous regions have both advantages and disadvantages as detailed in Table 1.1.

1.3.1.1. It would thus be seen that in the case of mountainous and sub-mountainous regions, the disadvantages outweigh the advantages of having a structure there. However, if it is necessary to locate it in these regions only for any other reason, then the structure will need to be designed and constructed properly so that maintenance problems are reduced to the minimum, if not altogether eliminated.

1.3.2. While the above listed disadvantages are absent in the structures located in trough, alluvial and deltaic regions, they have further advantages such as uniform flow, lesser percolation losses, more fertilising material in the water, etc. However, proper silt excluding measures at the headworks will need to be incorporated wherever necessary.

1.4. Whenever any diversion scheme is thought of, the possibility of having a weir should be first considered. Only when it is proved that it is not economically or otherwise not suitable, the barrag should be considered. Due to the cost of gates, etc., the cost of barrage is normally more than that of a weir.

TABLE 1.1

Advantages and Disadvantages of having barrage/weir in Mountainous and Sub-mountainous Regions.

Advantages	Disadvantages
(*i*) Water will be silt free	(*i*) There will be problems at the head works due to shingle. Proper arrangements for excluding shingle at the headworks will have to be made.
(*ii*) Due to availability of rocky bed at shallow depths, less foundation problems and hence lesser cost.	(*ii*) Since service area will start after some distance from the headworks, the idle length of canals will be more.
	(*iii*) Due to the many drainages in the regions, there will be more number of cross drainage works needed for aligning the canal.
	(*iv*) Due to steep slopes in the regions, the canal will have to be aligned with more number of falls.
	(*v*) Due to rolling of boulders, etc., there is likelihood of damages to the pier noses and wearing surface of the floor.
	(*vi*) As the structure will have to be designed to resist high velocities, better materials will have to be used in the construction and hence the cost will be more.
	(*vii*) Due to flashy nature of the rivers, protection on the downstream will have to be carefully evolved and implemented.

1.4.1. While working out the preference for a barrage or weir at a location the following points given in TABLE 1.2 in their favour need to be considered.

1.4.2. After considering the advantages and disadvantages, the type of structure i.e., barrage or weir may be decided.

1.5. According to the usage, the weirs can be classified as diversion weir, storage weir, waste weir and pick-up weir. According to the profile, the weirs can be divided into the following types :

(*i*) Vertical Drop Weir

(*ii*) Slope weir

- (*a*) Solid masonry weir.
- (*b*) Rockfill weir.
- (*c*) Glacis type weir.

(*iii*) Ogee or Parabolic type weir.

(*iv*) Buttress weir.

(*v*) Crib weir.

(*vi*) Syphon weir.

(*vii*) Trench type of weir.

TABLE 1.2

Points to be considered while selecting a barrage vis-a-vis weir.

Barrage	Weir
Advantages	
(*i*) Due to low crest of the undersluice and spillway, the afflux created on the U/S by the barrage is usually small.	(*i*) Where the natural banks are high enough to contain the affluxed water within the banks or when only low flood protective bunds would be necessary, a weir would be economical.
(*ii*) With barrage, better control over the river flow can be effected by judicious operation of the gates.	(*ii*) Due to the provision of gates only in the sluice bays, the operation would be easier and cost of the structure would be less. Wherever needed, the pond level could be conveniently raised by falling shutters up to about 2 metres.
(*iii*) Usually, a road bridge or railway bridge can be conveniently and economically combined with a barrage wherever necessary.	
Disadvantages	
(*i*) Cost of barrage is higher than that of a weir generally.	(*i*) Control over the river flow cannot be had fully as in the case of barrages.
	(*ii*) Chances of silting on the U/S are more.

1.5.1. A short description of the different types is presented below for information and their suitability or otherwise for particular conditions indicated. The various types indicate the development of weir sections.

1.5.1.1. *Vertical Drop Weir.* Vertical drop weir consists of a vertical masonry wall called weir or crest wall or body. In this type, the jet separates from the weir at the crest and falls freely on the downstream apron. This type of weir is usually suitable in sub-mountainous regions and particularly suitable for low falls unless it is founded on solid rock. *This is suited for hard clay* and consolidated gravel foundation. Generally, rectangular or trapezoidal sections are used. The crest is given a small rising slope of say 1 in 12 (See Figure 1.1). This diverts the stream leaving the weir in upward direction which has a double action of creating a back roller that deposits silt on the downstream side rendering the weir safe against scour and keeping the pond upstream in the immediate vicinity of the weir clear of silt deposit.

1.5.1.2. *Slope Weir.* In slope weir, slope is provided invariably on the downstream of the crest and on upstream also wherever needed. In this type of weirs, the jet of water remains in contact with the weir all the way. This type of weir is best suited where sliting of the pond is not desired and large size pebbles and even boulders are likely to roll down the weir. This can be conveniently constructed on soft sandy foundations like those in alluvial region. The three different types under the general classification of slope weirs are discussed below.

1.5.1.2.1. SOLID MASONRY TYPE

The solid masonry weir consists of a body wall, pucca apron on the upstream, pucca sloping masonry apron on the downstream and talus on both the upstream and downstream. This type of weir is usually constructed on fine sand foundation. In some cases, sloping aprons with one or more curtain walls are also provided. This type can be adopted in all conditions except at sites where the vertical drop is more than 5 metres. (See Figure 1.2).

1.5.1.2.2. ROCKFILL TYPE

After the name implies, boulders are laid to the required slopes on the upstream and downstream side with a few walls across the flow which make it more stable in the event of partial failure. The downstream slope is generally made very flat with the object of leading the current gradually and providing sufficient length of creep through the boulders. The principles of design of such type of weirs are not definitely laid down. As the water is free to release its surplus energy upwards in this type, uplift pressures need

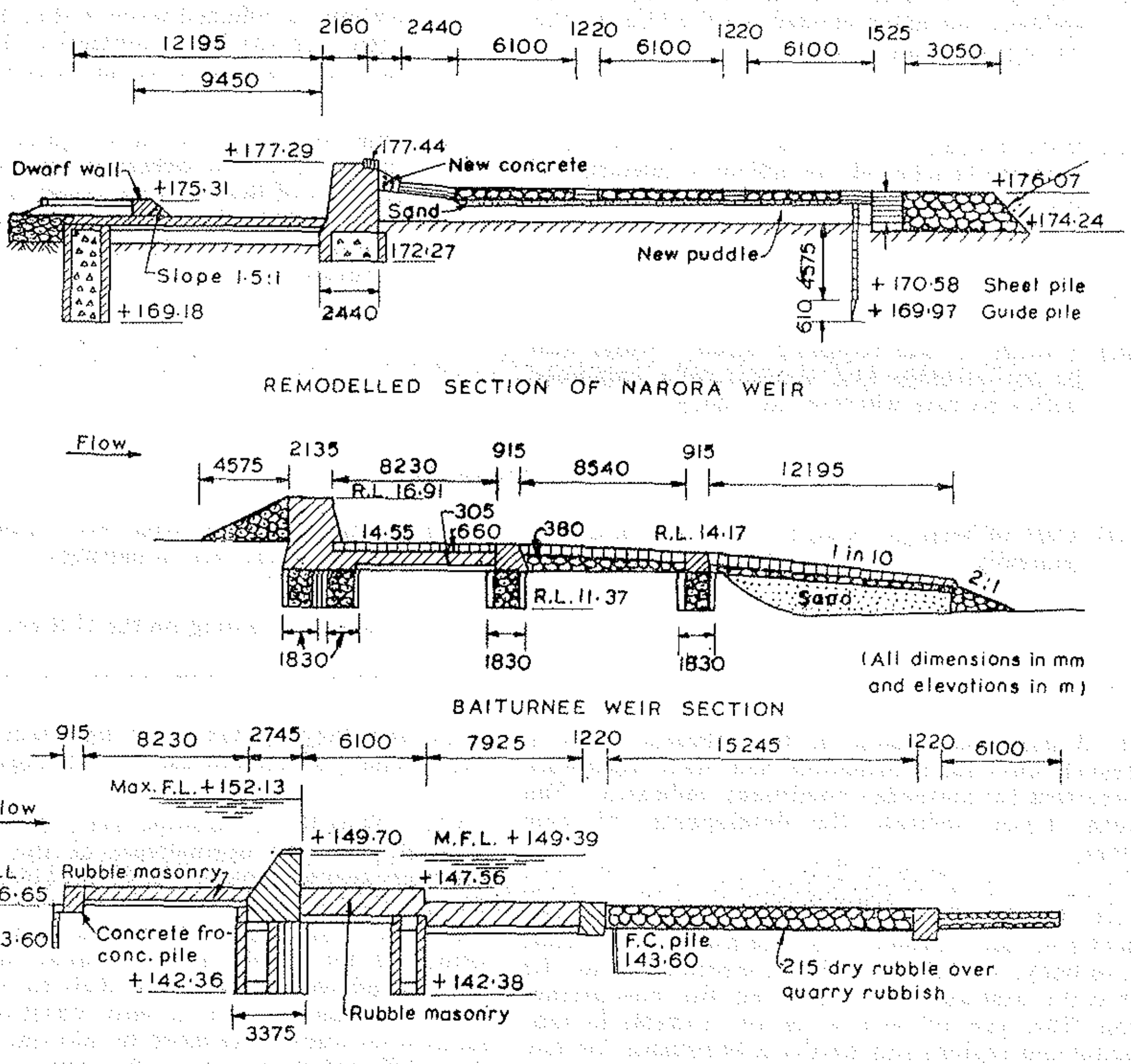


FIGURE 1.1 : Vertical Drop Weirs.

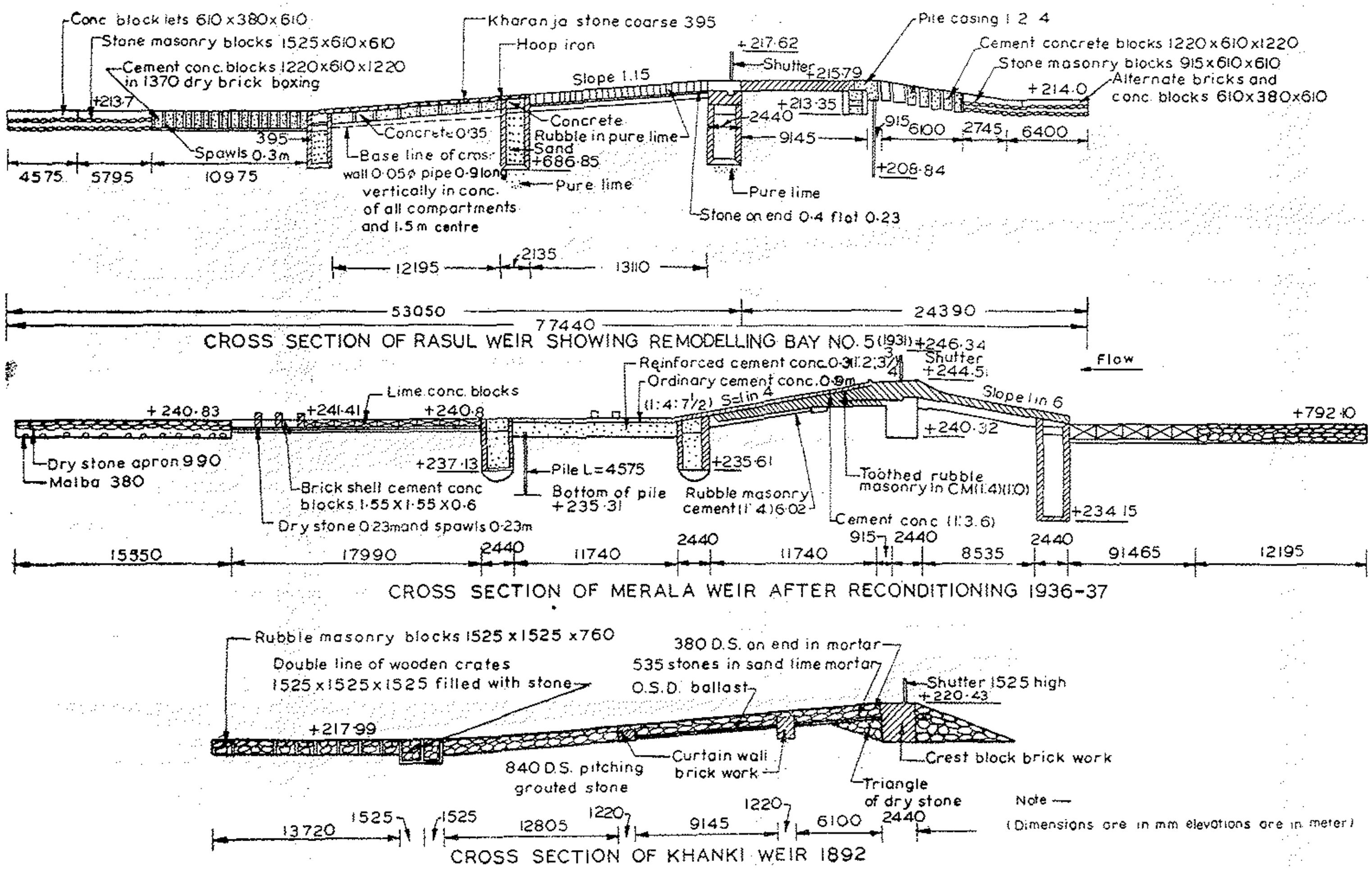
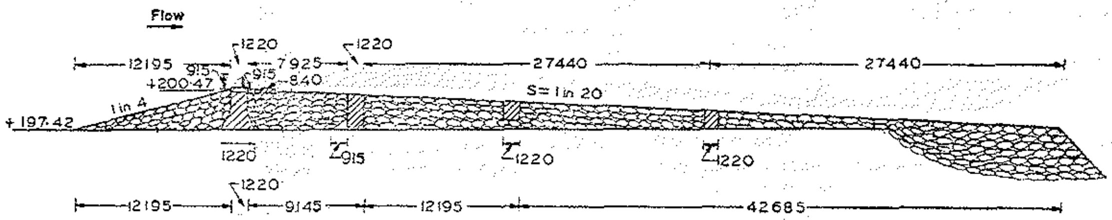
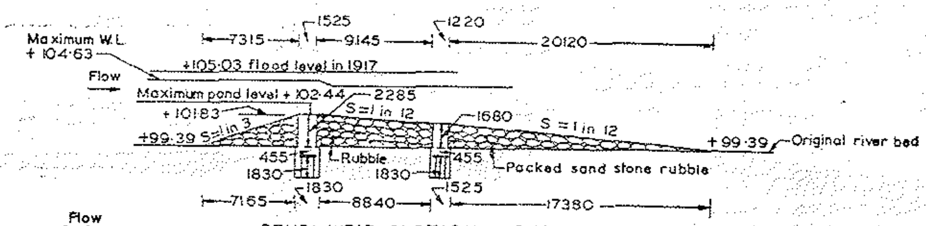


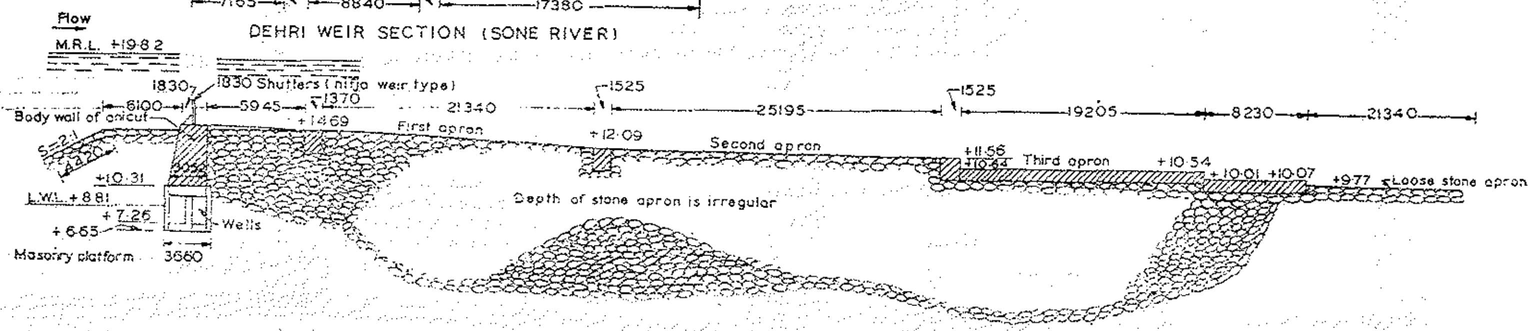
FIGURE 1.2 : Solid Masonry Weirs.



OKHLA WEIR SECTION AS COMPLETED IN 1897



DEHRI WEIR SECTION (SONE RIVER)



REMODELLED KRISHNA ANICUT SECTION (1949)

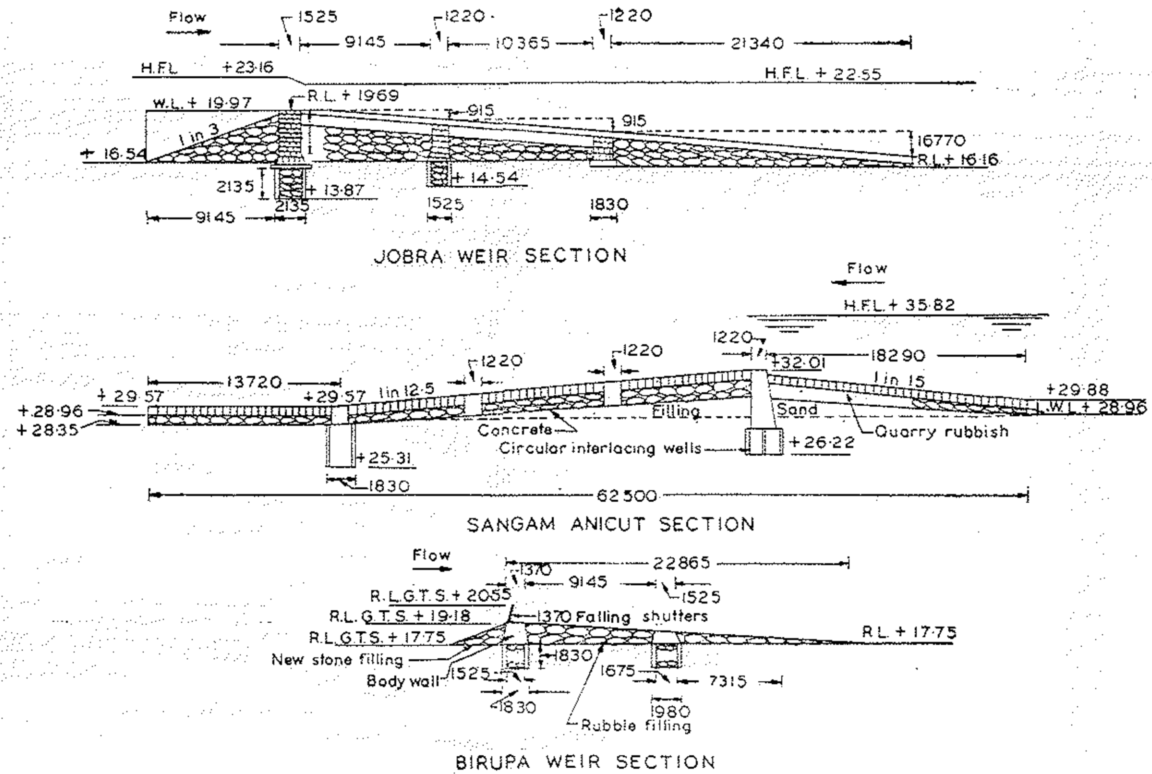
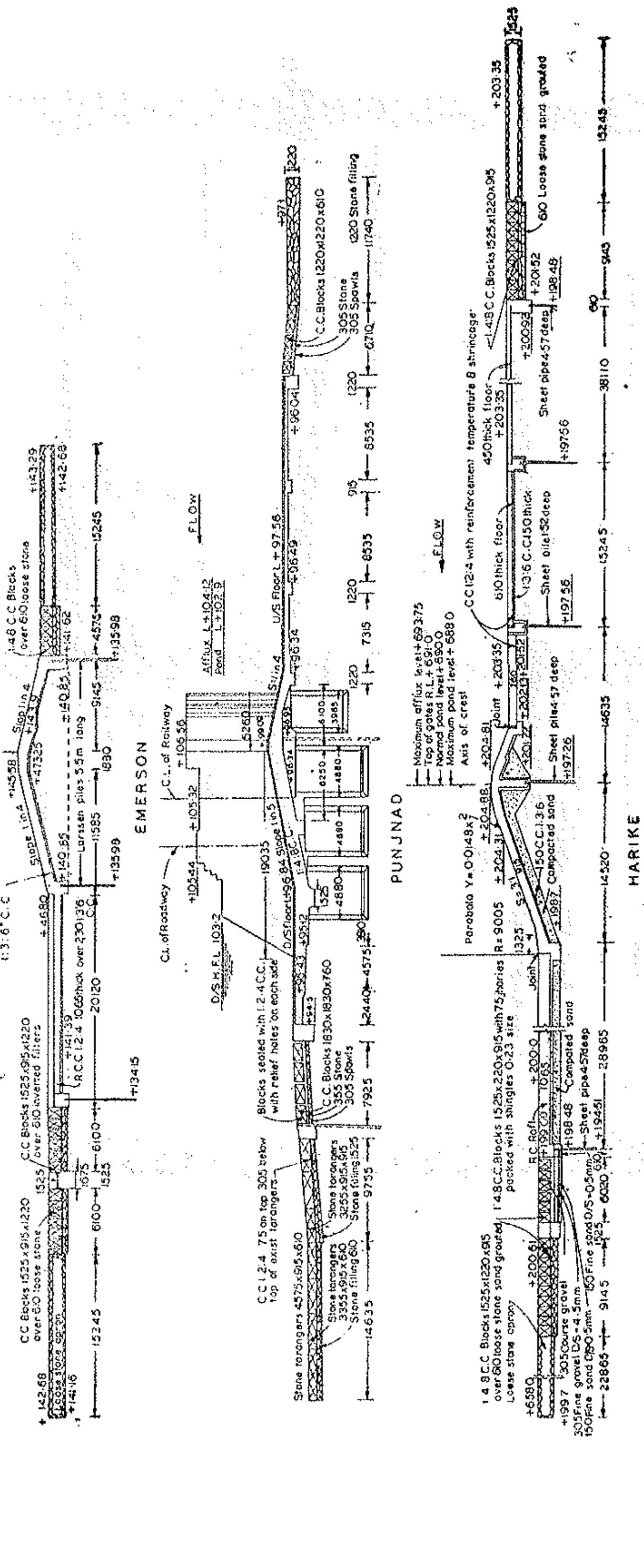


FIGURE 1.3 : Rockfill Weirs.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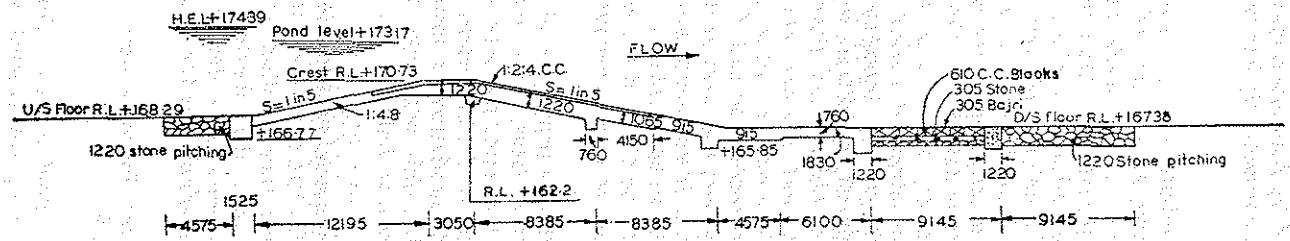

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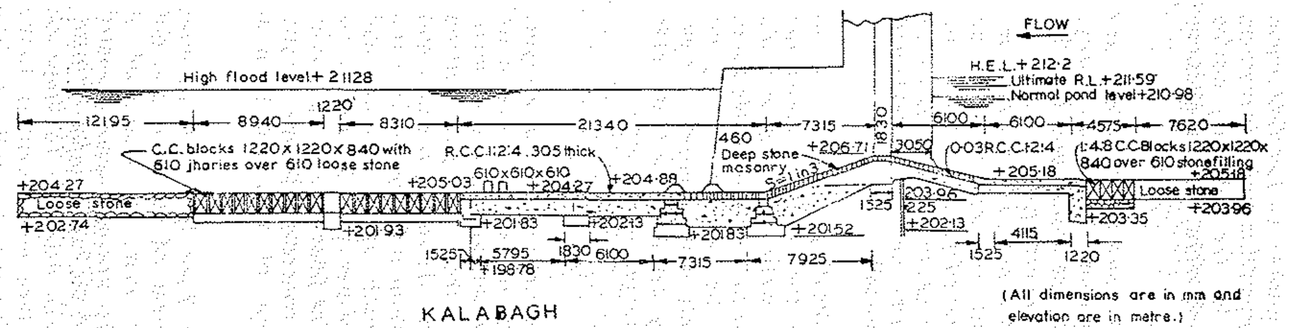


KALABAGH

FIGURE 1.4 : Glacis Type Weir and Sections.

not be considered. The rockfill on the upstream side may become impermeable in course of time by the deposition of silt and sand into the voids. This type of weir is particularly suitable to low differences of water levels between the upstream and downstream where velocities generated by the fall may not be excessive. It is also more suitable for mountainous regions.

The rockfill type of weirs constructed in India are founded on all types of foundations like sandy, clayey, and rocky. Rockfill weirs are the oldest type in India and are often remodelled by increasing their lengths or introducing curtain walls upstream and/or downstream. The large quantities of stone required both at the time of initial construction and subsequent refilling due to sinking, etc , preclude their use in many cases except where skilled labour and mortar are not available and stone can be obtained economically.

From Figure 1.3, indicating typical rockfill weirs, it may be seen that in the case of Krishna and Sangam Anicuts, the body walls and curtain walls have well foundations carried to a considerable depth below the river bed. It may be economical to grout the interstices of the body wall with concrete. However, this has to be done very carefully so that uplift pressures and consequent damages are avoided.

1.5.1.2.3 GLACIS TYPE

Glacis type of weirs, also referred to as Khosla's type, finding adoption now with the solutions for design are increasingly of weirs on permeable foundation propounded by Dr. A.N. Khosla and others. A typical section of this type has upstream and downstream glacis and horizontal floors in continuation of the glacis. Two and or more piles lines, one each at the upstream and downstream and some intermediate piles if found necessary are also introduced. The floor may be of gravity type or reinforced cement concrete raft type. Typical examples of Glacis type of weir are given in Figure 1.4.

1.5.1.3. *Ogee or Parabolic Type Weir* : If the sheet of water passing down the creat of the weir does not adhere to the surface of the weir, the likelihood of damages caused due to the partial vacuum created between the weir surface and the lower nappe of water is more. Hence it is always desirable to shape the crest and the downstream face corresponding to the profile of the lower nappe for the design flood. The shape of the falling jet over a sharp crested weir has become a standard form called the Ogee shape. Sometimes, a parabolic shape is also adopted, where generally a bucket is provided at the toe for the formation of hydraulic jump to dissipate the surplus energy. These types are suitable for greater heights of weirs on impermeable foundations like rock or firm clay. As the coefficient of discharge in the case of Ogee type is high, the length of such a type of weir required to pass the design discharge with permissible afflux will be lesser.

Typical examples of Ogee and Parabolic shaped weirs are given in Figure 1.5.

1.5.1.4. *Buttress Weir* : Solid masonry weirs require a considerable amount of building material which acts essentially only by its weight, whereas the strength of material frequently is hardly utilised at all. Where it is very difficult to procure these materials including transportation difficulties, the buttress type of weir originated by Ambursen may be employed to advantage since considerably less material is required for the same.

An Ambursen weir consists of water-tight facing of slabs resting on a series of triangular buttress. If a strong sound rock foundation exists, each buttress is founded separately. On a weaker or soft foundation, it may be desirable to place all buttresses on a common R.C. Slab or mattress. The buttresses of high weirs are braced with R C struts On a rocky foundation, the seepage under the weir is prevented by means of a concrete cut-off 1 to 2 m deep at the heel of the weir In loose soils, sheet piles are driven both at the heel and toe to safeguard against scour and undermining. Weep holes in the foundation slab, protected by inverted filter at the bottom would permit the escape of any water which may have found its way past the upstream sheet pile line and protect the slab against uplift. The weir is either provided with a deck only on the upstream side and flood waters are allowed to fall freely over the crest or it is constructed as a slope weir with a downstream face. The angle of upstream deck with the horizontal is usually between 38° and 45°. Near the Crest of the weir, air holes are necessary to destroy the partial vacuum created there.

The design principles of buttress weirs are similar to buttress dams. The weight of the deck of one bay and the water pressure on one bay are added to the weight of the buttresses. If the friction between weir and foundations is insufficient to prevent sliding, foundation slab with ribs may be provided or sheet piles driven to hold the weir securely.

The material required for a buttress weir may be 60 percent less than that for a gravity weir. However, this saving may be offset by much higher unit price of R C.C., but the shorter construction period for Buttress weirs may decide the issue. Buttress weirs are economical only for heights more than 5 m.

Typical Buttress weir sections are indicated in Figure 1.6.

1.5.1.5. *Timber Weir* : Timber weir, as the name indicates, *i.e.*, constructed with timber. Considering

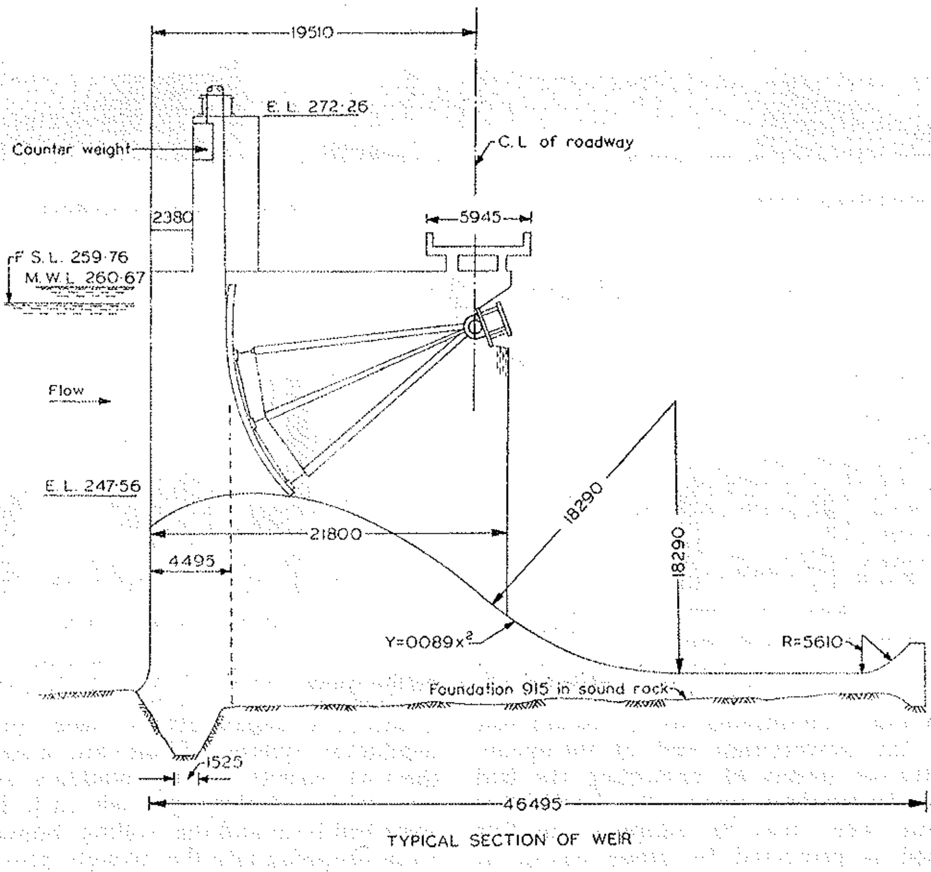


FIGURE 1.5 : Ogee Type Weir.

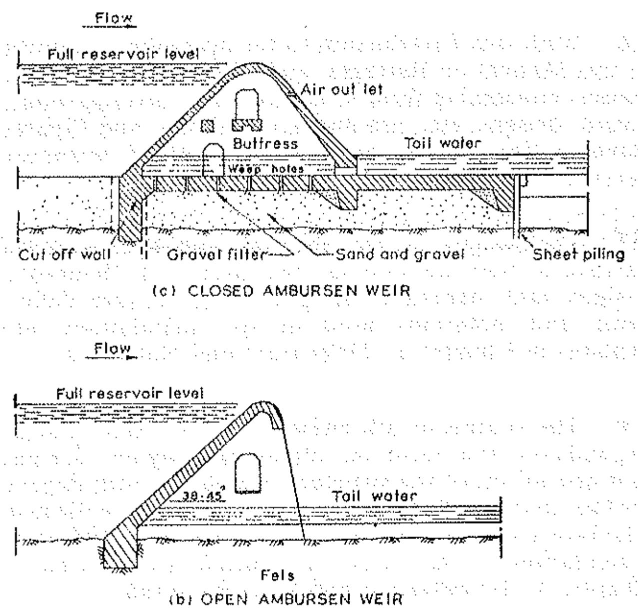


FIGURE 1.6 : Ambursen Type Weirs.

the quantity of timer required and the maximum life of about 30 years of timber, it is suitable in forest areas where timber is available in plenty, where there is difficulty in bringing other construction materials to the site and where the local inhibitants could be employed for construction.

A simple timber weir could be made of brush wood, round timbers and gravel as shown in Figure 1·7. Squared or round timber is used for the components of the weir.

If the bed consists of rock and sheet piling or piles cannot be driven, crib weir may be used. The cribes are constructed of round timbers, sometimes with transverse walls. The timbers are joined together at their intersections with clamps, nails or bolts. The cribs are filled and weighted with gravel and stones. Such weirs are initially permeable but due to silting up in course of time, they become tight.

The simplest type of timber weir for small heads consists of sheet piling driven between truss piles. The truss piles are braced by means of struts and the crest is formed by a timber cap. Scour is prevented

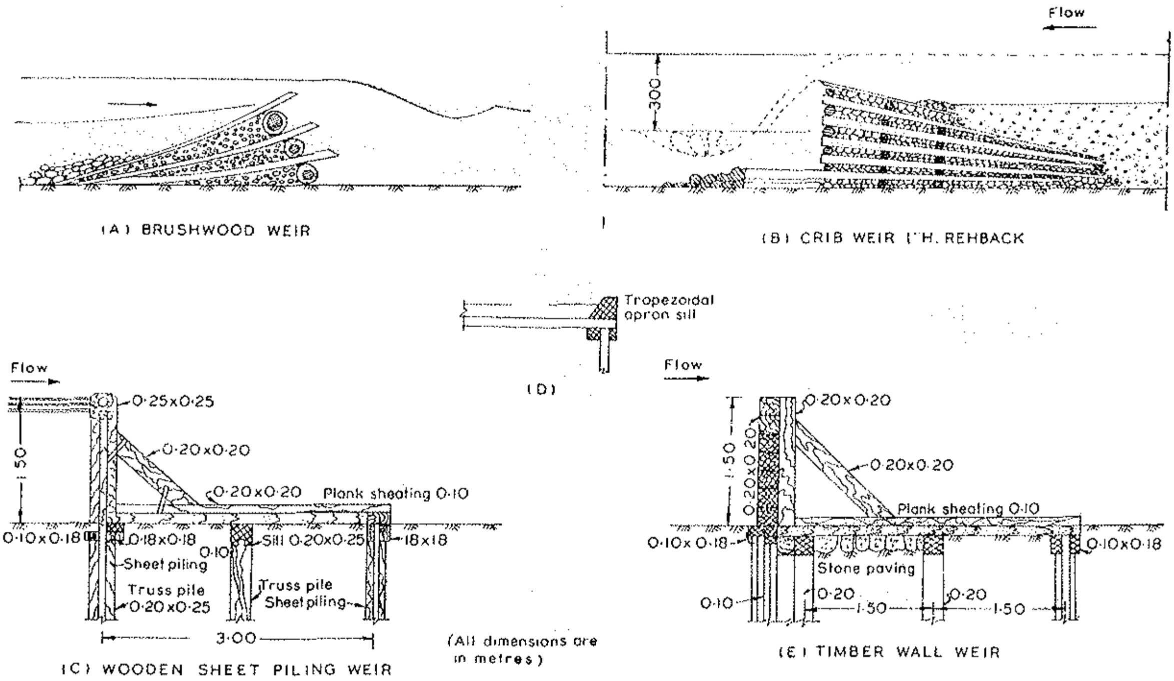


FIGURE 1.7 : Timber Weirs.

by a plank apron and undermining by a second row of sheet piling at the downstream end of the apron. An especially effective means of protecting the bed below the weir is a trapezoidal apron sill. For higher heads, timber wall weir may be adopted. In this type, the river bed is protected by stone paving at least 30 cm thick. A strong and particularly water tight timber weir of the slope type is also shown in Figure 1.8. Seepage under this weir is prevented by three parallel rows of sheet piling, the space between the piling being filled with clay placed and rolled in layers approximately 18 cms thick. The layers of clay are inclined gradually so as to make the last layer parallel to the plank deck. The clay must be placed so that the formation of voids is impossible. To prevent lateral seepage, the middle sheet piling wall is extended and keyed into both the banks. The timber joints in such a weir air also shown in the same Figure 1.8. The upstream deck is always shorter than the downstream deck. If neither ice nor drift is expected, the weir may be built up without an upstream deck in which case the weir consists only of two sheet piling walls, the upstream one carrying the crest cap. The planks of both the decks are butt jointed and covered with laths. Wooden abutments must be repaired frequently and for this reason stone masonry or concrete is used often instead. The bed at the end of the downstream slope must be protected from scour by rip rap or gravel.

1.5.1.6. *Trench Type of Intake* : In hilly reaches with very steep slopes of the river, sometimes, trench type of intake arrangements are made for feeding the water conductor system. These are generally used where the river carries rolling boulders and the banks are not stable. A sloping trash rack is installed at the river bed level and the rolling boulders are excluded from dropping into the trough provided below. The waterway of the trench type of intake is designed assuming about half the trash rack choked with boulders.

1.6. With this introduction to the diversion structures in this Manual on Barrages and Weirs, the different aspects concerning these structures like investigations, layout, designs, etc., are discussed briefly and Figures showing the details have been included wherever necessary.

1.7. The designs of the various components of the barrage or weir are done in two parts, viz., hydraulic designs and structural designs. The various definitions and notations used in the calculations are explains in Chapter 2—Definitions and Notations.

1.8. The success or otherwise of a diversion scheme depends on the location, alignment, layout, designs and operations of the structure which in turn depend on the investigations carried out and data collected. The investigations and data necessary for satisfactory formulation of the diversion scheme are detailed in Chapter 3—Investigation and Data Required.

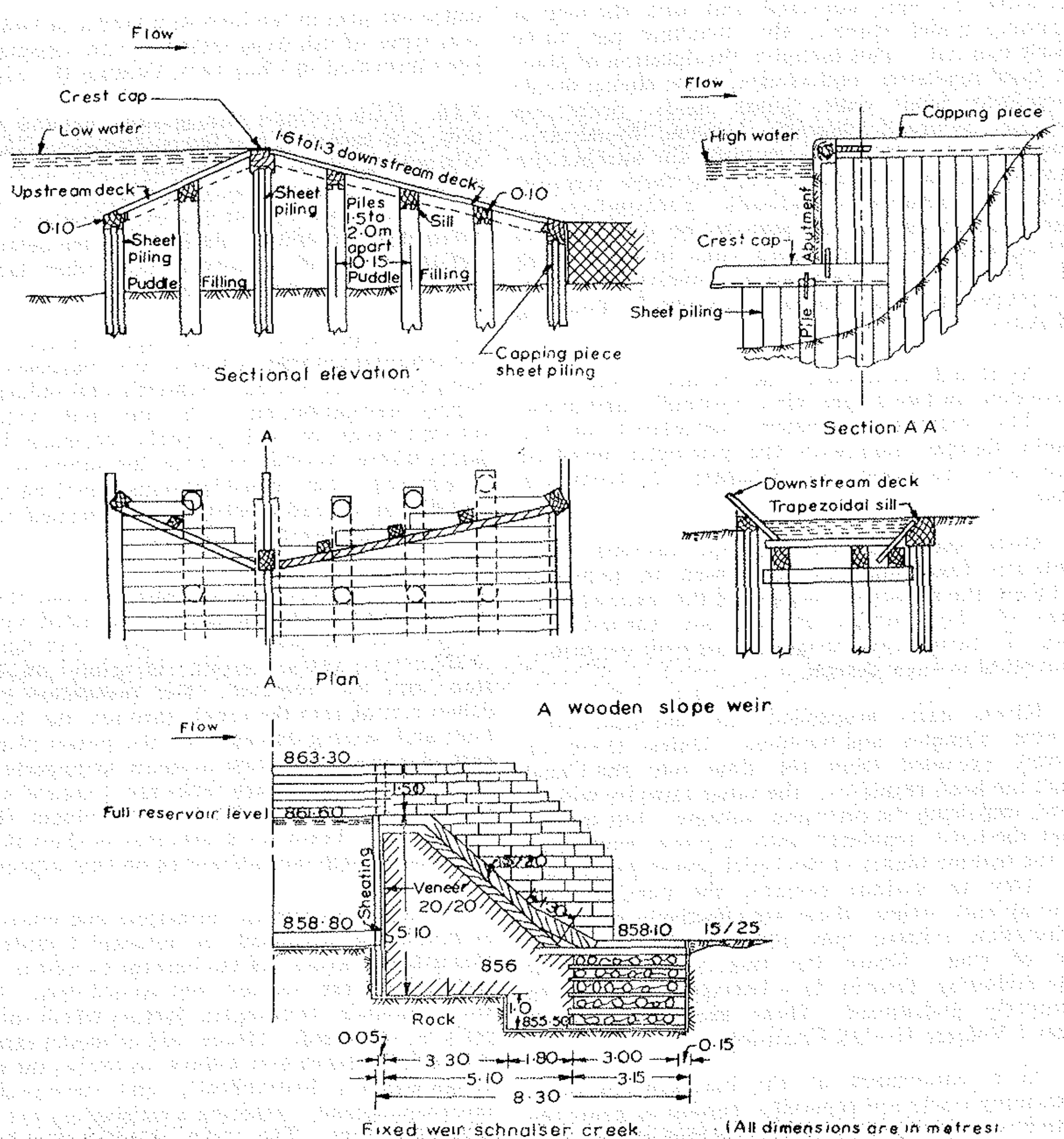


FIGURE 1.8 : Timber Weir.

1.9. After the necessary investigations are carried out and data collected, it is very important to locate the diversion structucture at a satisfactory alignment. Chapter 4—Locatioa and Alignment discusses these aspects.

1.10. As against different types of weirs, barrage are gated structures built across the rivers for diverting water for various purposes as enumerated in Para 1.0. Usually vertical steel gates are provided for the operation. A barrage could be constructed of stone masonry or concrete and this can be either of gravity type or R.C.C. raft. A barrage usually comprises the components like sheet pile or concrete cut off, pucca floor, piers, divide walls (provision depending on the necessity or otberwise), abutments, flankwalls, return walls wherever necessary, river training works whereever necessary, hoist bridge and hoist, upstream and downstream Pucca protection and loose stone protection, silt excluding arrangements whenever needed, etc. Wherever necessary, road and/or rail bridge would be provided over the barrage.

1.10.1. At the alignment chosen for the structure either from the data collected and with the help of preliminary model studies, the structure has to be properly laid out. This includes the location of abutment, head regulator, undersluice, river sluice, divide wall spillway, flank walls, guide bunds, protection works, etc., by proper planning. A layout found satisfactory for a site may not be suitable for another site due to the various conditions obtaining there. According to the satisfactory hydraulic performance of the structure, the layout will have to be decided and model experiments in this regard come in as a helping hand. The various considerations involved in planning a proper layout are set out in Chapter 5—Planning and Layout.

1.11. As already mentioned, the designs of the structure are done in two stages, viz., hydraulic and structural. The various parameters calculated in the hydraulic designs alongwith the principles involved thereof are discussed in Chapter 6—Hydraulic Designs.

1.12. Once the parameters of the various components are fixed and are confirmed by hydraulic model tests, the structural designs of the various components of the barrage or weir are carried out. Chapter 7—Structural Designs deals with the principles involved in their designs.

1.13. Rivers carry suspended silt and some rivers carry even shingles and boulders. Unless these are effectively excluded from the flow into the Canal through the head regulator, the canal capacity will be reduced, impairing the diversion scheme. Silt carried through the head regulator into a power canal may affect the turbine blades of the hydel power generating plant. Due to various reasons, the pond may get shoaled up and unless these are effectively removed, the diversion scheme may itself become unless in course of time. Hence the importance of silt and shingle excluding devices in a barrage or weir has to be properly understood. These are discussed in Chapter 1 Volume II—Silt Exclusion.

1.14. As a component of the barrages and weirs. river training works are generally needed to guide the river to flow axially through the barrage or weir and to check the outflanking besides providing favourable curvature of flow at the head regulator from the point of preventing sediment entry into the canal. The river training works are generally in the form of guide bunds, spurs or groynes, approach and afflux embankments. Chapter 2, Volume II—River Training Works deals with their layout, design considerations, etc.

1.15. For the development of pisciculture in some of the barrages and weirs, arrangements for migratory fish to move from upstrem to downstream and vice versa have to be made. This is required from environmental considerations which is necessary from the socio-economic point of view. The arrangement is called fish pass in the form of a ladder or lock. The different types of fish pass, their location, designs, etc., have been discussed in Chapter 3, Volume II—Fish Pass.

1.16. With Indland Navigation gaining importance now, it is necessary that the possibilities of having suitable arrangements at the barrages and weirs should be explored to cater for navigation facilities wherever practicable and potential for it exists Chapter 4, Volume II Navigation Facilities deals with the details of navigation locks and channels and other facilities like jettys, slipways for repairs, etc.

1.17. The purpose of a barrage or a weir is to divert the required amount of water for purposes of irrigation, power generation, domestic and industrial water supply, navigation, etc. The diverted waters are fed into the canals through properly designed head regulators where regulation over the supply of water can be effected. The considerations involved in location and design of head regulators are discussed in Chapter 5 Volume II—Head Regulators.

1.18. The control over the flow of water through the barrage and head regulator is effected by properly designed service gates. For repair and maintenance of the service gates, certain emergency measures like stop logs are required. For prevention of floating debris getting into the canal through the head regulator and causing damages to the power plants in the case of power generation schemes, arrangements at the head regulator like trash racks are required to exclude the floating debris. Chapter 6 Volume II—Gates, Stoplogs and Trash Rack describes in short the various types and design considerations on this aspect.

1.19. Based on certain principles and considerations as would be discussed in relevant Chapters of this Manual, the layout of the barrage or weir is prepared and designs are carried out accordingly. But there are a number of complex factors which cannot properly be visualised. Hence aid of model experiments is invariably taken now-a-days to evolve the optimum design which is hydraulically and structurally sound and econonmical, ensuring a satisfactory performance of the structure. The data required for laying the hydraulic model before testing, the various terms of reference for the hydraulic model tests, and their utility, etc ; have been discussed briefly in Chapter 7, Volume II—Model Tests.

1.20. The designs of barrages and weir are based on certain principles, both empirical and derived. However, due to various parameters involved, the designs may sometimes be oversafe or deficient in some respects. For this, if the performances of these parameters are known through some means, the designs can be improved upon, both for safety and economy. Instrumentation plays a good part in this. Chapter 8, Volume II—Instrumentation deals briefly with the different instruments that could be conveniently installed in the diversion structures for collection of data and analysis thereof.

1.21. The various considerations for sound construction of the diversion structure are detailed in Chapter 9, Volume II—Construction.

1.22. Even a well designed barrage or weir may pose problems leading to failures if the operation and maintenance of the structures are not done promptly and properly. Inspection of barrage or weir is necessary to obviate the possibility of prevention and/or extension of damages. Regulation of the gates before, during and after monsoon has to be carefully planned and carried out. It is also very important to keep a history of the headworks to know about the behaviour and overall performance, repairs carried out, etc. All these aspects are discussed in Chapter 10, Volume II —Maintenance and Operation.

1.23. When a barrage or weir is constructed across a river, it is to be understood that the natural flow of the river is being interferred with and due to this, there will be a change in the regime of the river which is likely to cause aggradation or retrogression of the bed and consequences thereof, etc. Hence before planning a barrage or weir, it is also necessary to study the effect of the structure on the regime of the river which will help in designing the structure safely at a properly selected site. The effect on the regime of the river has been discussed briefly in Chapter 11, Volume II.

1.24. In spite of the best efforts for a safe design and good construction, due to unforeseen parameters like flash floods, bad or poor operation, maintenance etc., sometimes there do occur failures of barrages and weirs. The different causes for damages and the data required to be collected in such cases for further analysis and remedial measures are discussed in Chapter 12, Volume II—Failures of Barrages and Weirs.

1.25. For building a safe structure, it is also necessary that materials of good specifications are only used and strict quality control is maintained.

1.26. Due to climatic conditions and nature of the sites in which the diversion structure is located, there might arise some special problems. These may occur during construction and design stages also. Some special problems usally faced in diversion structures are discussed in Chapter 13, Volume II.

1.27. This Manual on Barrage and Weirs is not an exhaustive one. Only the different aspects related to the diversion structures have been discussed in a concise manner. For a detailed study of all the aspects, the reader is advised to refer different text books, publications, standards, papers, etc. For convenience, a short Bibiolgraphy on the subject which will be of use to the reader is presented in Chapter 14, Volume II

CHAPTER 2

Definitions and Notations

2.0. Introduction

In this Manual on Barrages and Weirs, a number of terms relating to the discipline of diversion structures and also a number of notations are used. While these have been explained in the relevant paras where they occur, most of them have been compiled and given in this Chapter for convenience of study.

2.1. Definitions

The various terminologies adopted in this Manual have the following definitions.

2.1.1. *Abutment*

Retaining wall constructed at the end of the waterway of barrage/weir to retain the backfill, protect the banks from erosion, support load from superstructure and confine the flow to the desired waterway at the structure.

2.1.2. *Afflux*

The difference in water level between upstream and downstream of a barrage/weir under free flow condition as a result of construction of the structure on the river.

2.1.3. *Afflux Bund*

An embankment or dike designed to ensure that the structure is not outflanked during flood flows and in some cases acting as embankment to prevent flooding of the country side due to rise in water level.

2.1.4. *Aggradation of Level*

An increase of rise in specific levels of the bed (the bed level at specific discharge) of a channel at any site.

2.1.5. *Alluvial River*

A river which flows through deposits created by itself and is constantly building its flood plain by overflowing.

2.1.6. *Apron*

A protective layer of stone or other material provided in the bed of the river where it is desired to prevent erosion.

2.1.7. *Backwater Curve*

The upstream longitudinal profile of the surface of water in a stream or conduit from a point where such water surface is raised above its normal level by a structure such as a weir/barrage or dam.

2.1.8. *Barrage*

A barrier provided with a series of gates across the river to regulate water surface level and pattern of flow on the upstream and for other purposes, distinguished from a weir in that it is gated over its entire length and may or may not have a raised sill.

2.1.9. *Bay*

One of the main divisions of the diversion structure such as spillway, undersluice, regulator, etc., between two piers or a pier and abutment or a pier and divide wall.

2.1.10. *Bed Material*

The material, the particle size of which are found in appreciable quantities in the shifting portions of the bed.

2.1.11. *Block-outs*

Temporary recesses provided in the civil structrure to facilitate proper embedment of steel fixtures for gates, trestles, etc., and which are concreted after their fixing.

2.1.12. *Boulder Stage of River*

The reach of river when it emerges from the hills with a well defined cross-section and non-submersible banks on either side.

2.1.13. *Chute Blocks*

Staggered R.C.C. blocks provided at the toe of the downstream glacis for energy dissipation.

2.1.14. *Coffer dam*

A temporary structure constructed to exclude water from work site during construction.

2.1.15. *Concentration Factor*

The factor by which the discharge per unit length of a barrage or weir (assuming uniform distribution) is required to be multiplied to get the design discharge per unit length for designing its various elements. This factor is required to be applied since flow in the different bays may not be uniform due to various factors.

2.1.16. *Construction Joint*

A joint occurring in a structure composed of homogeneous material, such as earth or concrete, along a plane or surface, formed by cessation of placing of material for a time, such as overnight or for several days.

2.1.17. *Contraction Joint*

A Joint provided to localise and minimise development of cracks due to drying shrinkage and thermal variations.

2.1.18. *Crest*

The line or area defining the top of the weir/barrage.

2.1.19. *Cut Offs*

Lugs either of R.C.C. masonry or of steel sheet pile, provided at the bottom of the structure to protect the structure against scours and possible piping due to excessive exit gradients of the seepage flow underneath the structure.

2.1.20. *Design Flood*

The discharge of river with certain frequency of occurrence (such as 100, 200, 500 years, etc) for which the waterway and other parameters of the diversion structure are designed.

2.1.21. *Dewatering*

Lowering the water table to facilitate construction of the barrage/weir substructure and items to be done in fairly dry condition and to prevent free flow of particles below the foundation.

2.1.22. *Differential Head*

The difference in water level between upstream and downsteam water surfaces of the structure or the difference in water level on either sides of a pier or divide walls.

2.1.23. *Divide Wall*

Wall constructed usually at right angles to the axis of the barrage or weir extending well beyond the main structure to separate the under-sluices, river-sluices and spillways into independent units for facilitating regulation.

2.1.24. *Dominant Discharge*

The discharge which is large enough in magnitude and is of sufficient frequency of occurrence to have a dominating effect in determining the size and characteristics of the river course, channel and bed.

2.1.25. *Embayment*

The area within the swing of the bend of a river. Also a local recession of a river bank due to erosion *i.e.*, a bite taken out of a river bank.

2.1.26. *End Sill*

Raised sill provided at the end of the stilling basin for energy dissipation.

2.1.27. *Erosion*

The lowering and wearing of the land by weathering and transportation under the influence of gravity or water.

2.1.28. *Exit Gradient*

The upward seepage force per unit volume of percolating water through foundation soil at the tail end of a weir or barrage, tending to lift up the soil particles if it is more than the submerged weight of a unit volume of the latter. It is also defined as the hydraulic gradient of emerging stream lines at the end of an impervious apron.

2.1.29. *Expansion Joint*

A joint provided to localise and minimise fixed points to permit longitudinal expansion and contraction when changes in temperature occur and to permit vertical movement where differential settlement is anticipated.

2.1.30. *Filer*

A layer or combination of layers of graded pervious materials designed and placed in such a manner as to provide drainage and yet prevent the movement of soil particles with seepage water.

2.1.31. *Fish Ladder/Lock*

Device provided in the diversion structure to facilitate the passage of fish from upstream to downstream and vice versa.

2.1.32. *Free Board*

The vertical distance between a specified water surface and top of the component of a structure under consideration.

2.1.33. *Friction Block*

Staggered R.C.C. blocks provided in the stilling basin for energy dissipation.

2.1.34. *Froude Number*

The dimensionless number obtained by dividing the mean velocity by propagation velocity of an infinitely small surface wave (the square root of the product of the depth and the acceleration of free fall).

2.1.35. *Glacis*

The sloping protion of the floor upstream and downstream of the crest.

2.1 36. *Gauge-Discharge Curve*

The curve indicating the various stages of the river for different values of discharge.

2.1.37. *Guide Bund*

A protective and training embankment constructed at the side of a weir/barrage to guide the flow through the waterway.

2.1.38. *Head Regulator*

A structure built upstream of the diversion structure to divert, control and regulate water supplies from the pond.

2.1.39. *Hydraulic Jump*

The sudden and usually turbulent passage of water from low level below critical depth to high level above critical depth during which the velocity passes from supercritical to subcritical.

2.1.40. *Hydrograph*

A graph showing the stage, volume of flow, velocity, sediment concentration or sediment discharge or some other feature of flowing water with respect to time at a given place.

2.1.41. *Looseness Factor*

The ratio of the overall length of the weir/barrage provided to theoretically computed minimum stable width of the river at the design flood obtained by using Lacey's equation.

2.1.42. *Meander*

A meander in a river consists of two consecutive loops, one flowing clockwise and the other anticlockwise.

2.1.43. *Modulus of Subgrade Reaction*

The load distributed on unit area of an elastic medium which creates unit deflection.

2.1.44. *Pier*

Concrete or masonry structure constructed over the waterway for supporting bridge decking, gates and hoist operating mechanism.

2.1.45. *Pitching*

A protective covering of properly packed or built-in-materials on the surface of the earthern sides (side pitching) to protect them from the action of water.

2.1.46. *Pond Level*

The level of water immediately upstream of the barrage/weir required to facilitate withdrawal into the canal or for any other purpose.

2.1.47. *Regime*

The condition of a stream or its channel as regards their stability.

2.1.48. *Retrogression of Level*

The lowering of specific bed levels of a channel.

2.1.49. *River Slucies*

A set of sluices similar to the undersluices located in between the undersluice and spillway bays and separated from them by mean of divide walls.

2.1.50. *River Training*

Engineering works including artificial plantations (with or without the construction of embankment) built along a river or section thereof in order to direct or to lead the flow into a prescribed channel.

2.1.51. *Scour*

The removal of material from the bed of a channel by flowing water.

2.1.52. *Sediment*

Fragmental material transported by, suspended in or deposited by a flowing channel without any regard to size.

2.1.53. *Seepage*

A slow movement of water due to capillary action through pores and interstices of unsaturated packed soil material into or out of a surface or subsurface body of water such as river, canal, etc.

2.1.54. *Sheet Pile*

Steel sheet pile provided to cater for cut-off, scours, etc.

2.1.55. *Silt*

A fine grained soil with little or no plasticity.

2.1.56. *Silt Excluder*

A device by which silt is precluded from entering the canal, consisting ususally of a series of R.C.C. tunnels located in front of the head regulator and at right angles to the axis of the barrage or weir

2.1.57. *Silt Factor*

A factor '*f*' in the Lacey's formula and is given by the following equation in regime channels :

$$f = 1{\cdot}76\sqrt{M_r}$$

where M_r=average particle size in mm.

2.1.58. *Specific Energy*

The energy of stream per unit weight referred to its bed ; namely, depth plus velocity-head corresponding to mean velocity.

2.1.59. *Stilling Basin*

A short length of paved channel in the exist course of an outlet structure or below a spillway in which all or part of the energy of flowing water is dissipated and water discharged into the downstream channel in such a manner to prevent damage to the structure or dangerous scour of bed or banks of channel.

2.1.60. *Stone Reserve*

A quantity of stone kept as reserve on guide bunds, spurs or groynes for emergency use to prevent deep scour occurring and endanger the safety of the structure.

2.1.61. *Stoplogs*

Fabricated structural steel or wooden units utilised for temporary closure of any bay of the barrage or regulator in order to facilitate repairs of the gates and other components of the bay.

2.1.62. *Sub-Critical Flow*

The flow in which the Froude number is less than unity and surface disturbances can travel upstream.

2.1.63. *Super Critical Flow*

The flow in which the Froude number is greater than unity and surface disturbances will not travel upstream.

2.1.64. *Toe wall*

A shallow wall construeted below the bed or floor level to provide footing for the sloped pitching or the face of an embankment.

2.1.65. *Trash Rack*

Metallic racks in front of a head regulator to screen out floating materials.

2.1.66. *Unit Hydrograph*

Hydrograph of storm run-off at a given point on a given stream which will result from an isolated rainfall excess of unit duration occurring over the contributing drainage area and resulting in a unit of run-off.

2.1.67. *Uplift*

The upward hydraulic pressure in the pores of a body (pore or interstitial pressure) or on the base of a structure.

2.1.68. *Velocity of Approach*

The mean velocity in the conduit or stream immediately upstream of a barrage/weir.

2.1.69. *Waterway*

A sectional area or an amount of opening (vent) for flow of water through weirs/barrages, head regulators, etc.

2.1.70. *Weir*

Ungated barrier across a river to raise the water level in the course.

2.2. Notations

The notations indicated below have generally been adopted unless mentioned otherwise :

a = semi-major axis of ellipse (See Chapter on River Training Works).

b = semi-minor axis of ellipse (See Chapter on River Training Works).

D = maximum anticipated scour depth below water level assumed at low water level (LWL)/Floor

f = Lacey's Silt factor

F = free-board

L = total waterway of barrage or weir

m = weighted mean diameter of bed material in mm.

P_W = Lacey's waterway in m.

Q = Design flood discharge in m^3/s.

S_s = Specific gravity of the particle

T = Thickness of stone pitching in m.

V = Mean velocity in m/s

W = Minimum weight of stone in kg

θ = angle of face slope in degrees

A_t = Steel reinforcement

K = Modulus of Subgrade reaction

R = Scour depth measured below High Flood Level

D_1 = Depth of water before hydraulic jump

D_2 = Depth of water after hydraulic jump

E_{f1} = Energy of flow just upstream of the point of formation of standing wave

E_{f2} = Energy of flow just downstream of the point of formation of standing wave

H_L = Head loss, *i.e.*, difference between the upstream and downstream total energy line values

V_a = Velocity of approach

q = intensity of discharge in cumecs/m

g = acceleration due to gravity in m/sec^2

α_h = Coeff. of horizontal acceleration due to earthquake.

α_v = Coeff. of vertical acceleration due to earthquake

G_E = Exit Gradient

b = length of barrage/weir section

d = depth of cut off (with suffixes wherever necessary)

H = Head of water

φ_C & φ_E = percentage of pressure at the top of sheet pile (with suffixes wherever necessary)

φ_D = percentage of pressure at the bottom of sheet pile (with suffixes wherever necessary)

2.2.1. Other notations have been explained in the Chapters wherever they occur.

CHAPTER 3

Investigations and Data Required

3.0. Introduction

After the necessity of constructing a barrage or weir has been established, for an economical and satisfactory design of the diversion structure and its canal system, proper investigations are required to be carried out and necessary data collected in a systematic way. The investigations are needed to be carried out without any laxity as the continued safety and efficient functioning of the diversion structure and integrating it with the master plan of the basin development depend on the data collected and analysed based on such investigations. Proper investigations are necessary to avoid a lot of delay and waste of time that generally occur in preparing design on an inadequate or misleading information. The designs based on uncertain and scanty data have often to be repeated and revised. It has been experienced that not only the designs have to be changed but sometimes even the site has to be abandoned due to various aspects such as poor foundation strata, configuration of river, etc. Besides, the structure designed to be safe based on an inadequate data may suffer an unexpected serious structural damage. Money spent in collecting accurate information before designs and construction are undertaken forms a very small part of the total cost of the work, but has its great value in preparing safe and economic designs speedily. The investigations are generally carried out in two stages, namely (*i*) Preliminary and (*ii*) Detailed. These are discussed in subsequent paras.

3.1. Preliminary Investigations

Preliminary Investigations are carried out on aspects of available maps, regional and site geology, foundation strata, available hydrogical data, water requirement, available construction materials and communication to the proposed site of work. These are further discussed in succeeding paras. Based on these preliminary investigations, all possible sites for the location of the barrage or weir should be marked on the available maps. After a site inspection and consideration of topography and others, it should be possible to eliminate some of the sites selected. Further sub-surface explorations and collection of hydrological data at the remaining sites would help in considering the merits and demerits of the remaining selected sites from which the sites can be graded according to their relative suitability or a final selection of the site for locating the barrage or weir can be made.

3.1.1. *Study of the Available Maps*

Study of the available maps, both old and latest, would give a general idea of the location and topography of the area from which possible sites for location of the diversion structure could be marked. Comparison of the maps would indicate the stability of the river at the proposed sites.

3.1.2. *Regional and Site Geology*

A critical study of the regional and site geology of the possible selected sites would indicate the presence or otherwise of adverse geological formations, such as faults, fractured zones, shear zones, solution cavities, etc. This would enable the designer either to eliminate a few sites or to determine the economics necessitated by the treatment required to take care of the adverse geological formations.

3.1.3. *Study of Foundation Strata*

A study of the foundation strata at the possible sites by means of existing nearby deep cuts and wells, project reports of nearby areas, trial pits and bore holes wherever necessary would enable us to determine the type of structure necessary at each of the possible sites and hence the economy involved.

3.1.4. *Study of Available Hydrological Data*

For a correct assessment of the water potential at the various sites, it is very important to make a proper study of the available hydrological data such as rainfall records in the catchment, river gauges and discharges, peak flows, etc. This would enable us to plan the canal system including the extent of service area, augmentation of the river supplies from another basin if necessary including ground water supplies and the diversion structure itself for its safety and economy. Hydrographs at the selected site should be prepared indicating the discharge during different months so that the minimum and average discharge during different months are assessed. Normal and maximum flood discharges should also be assessed for the design and regulation of the barrage.

3.1.5. *Assessment of Water Requirement*

It is necessary to make an assessment of the water requirements including possible future demands for irrigation, hydro power, drinking or industrial supply etc,, so that the headworks could be suitably designed with the necessary feeding levels. This will also help in planning for the conjunctive use of surface and sub-surface water available in the service area.

3.1.6. *Availability of Construction Materials*

The type of structure and economics thereof at the different sites for locating the head works or diversion structure would depend on the availability and quality of the construction materials in the vicinity of the sites. Hence a correct assessment of the same is necessary.

3.1.7. *Communication to the Site of Work*

While the choice of the final site for locating the structure should be made mainly from considerations of engineering and geology, due consideration should also be given for communication works for easy accessibility and economic transportation of materials to the site of work.

3.1.8. *Climate of the Region*

As the water availability, evaporation and construction techniques and period to be adopted depend on the climatological variations of the region in general and the work area in particular, data regarding the climate of the region needs to be studied.

3.2. Detailed Investigations

From the analysis of the dada obtained during Preliminary Investigations, the final selection of the site for locating the diversion structure with its head regulators can be judiciously made. For collecting the data necessary for the design of the structures at this site, detailed investigations need to be carried out. These should cover (*i*) detailed topographical survey (*ii*) collection of hydrological, sediment and evaporation data (*iii*) surface and sub-surface investigations including laboratory tests (*iv*) diversion requirements (*v*) construction materials (*vi*) communication system and (*vii*) other miscellaneous studies. These are discussed in the following paras.

3.2.1. *Detailed Topographical Survey*

For detailed topographical survey, the following are required to be carried out :

(*a*) An index map of the area has to be prepared. This should show the catchment of the entire river valley upstream of the selected site and should also show salient features like important irrigation works in the vicinity, road and railway crossings, hydrological stations, wet and dry fields, gardens, temples, grave-yards, villages, etc., which may come under submergence if any due to the pond created. The map may preferably be drawn to a scale of 1 in 4000.

(*b*) A contour plan of the site of barrage or weir has to be neatly prepared. The contour interval should not be more than 0·5 m and should cover elevation of at least 2·5 m above the high flood level. The contour plan shall extend to about 5 km on the upstream and downstream of the site and upto an adequate distance on both the flanks upto which the effect of pond is likely to extend. The plan should clearly indicate salient features like tributaries and their confluence with the river, firm banks, rock outcrops, deep channels, large shoals and islands, deed pools, important land marks, marginal erosions, etc. The length of survey may depend on the configuration of the river. This could be shortened or increased depending on the straight reach or meandering nature of the river. In case of meandering river, the length of survey should cover at least two fully developed meanders on the upstream and one of the downstream of the axis of the diversion structure. The plan may preferably be drawn to a scale of 1 in 4000. The requirement of survey lengths for detailed model studies is discussed in the chapter on Model Studies.

(*c*) Cross-sections of the river have to be taken at the axis of the barrage or weir at the proposed site and at regular intervals, both on the upstream and downstream of the axis. The interval shall be 200 m upto at least 600 m from the proposed site. In addition to these, cross sections of the river on the upstream side may be taken at 2 km intervals up to the distance to which the back water effect of ponding is likely to extend. The intervals for taking the cross section may be reduced if the topography indicates appreciable fall in the river slope. The cross-levels in the river bed may be spaced at 10 to 30 m depending upon the topography of the river. The cross-sections should generally be extended on both the banks upto about 2.5 m above the high flood level. It should be such that a proper layout of guide bunds and afflux bunds, wherever necessary, could be planned. In hilly streams, the cross-section intervals should be closer. The requirements of cross-section and longitudinal section in respect of model studies are discussed in the chapter on Model Studies.

(*d*) Longitudinal section of the river along the deep current should be taken for a distance of about 5 km on the downstream side and about 15 km on the up streams side. If the back-water effect is likely to extend beyond 15 km longitudinal section should be taken upto that distance. The water levels observed all along the section should be marked on the section.

3.2.2. *Hydrological Data*

The success or otherwise of the diversion scheme depends to a great extent on the accuracy and availability of hydrological data. These are required for assessing the available weekly and monthly run-off and for computing the flood for which the structure has to be designed. For these purposes, it is very important to collect the following data for further studies.

(*a*) Daily rainfall data of all rainfall gauging stations in the catchment area and also around the catchment area wherever available. The data should be collected for as many years as possible. Data regarding storms in respect of successive positions of the centre of the storm on the catchment should be collected. Storms causing peak discharges should be separated for unit hydrograph analysis.

(*b*) For working out unit hydrogrph, flood hydrographs for isolated rain storms are to be collected.

(*c*) If adequate data is not available, synthetic hydrograph has to be developed. For this purpose, catchment characteristics, such as shape, slope, orientation, drainage systems and infilteration capacity will have to be collected.

(*d*) For working out the design floods by frequency analysis, peak flow data for the river for as many years as possible should be collected.

(*e*) Daily stage and discharge data of the river at or near the site for as many years as possible should be collected. The data should cover both monsoon and non-monsoon periods. In case the gauge and discharge data at the proposed site are not available, the hydrological observations at the site should be immediately started so that the calculated and observed values could be correlated. Two gauges on the upstream, one at the axis and two on the downstream may be set up for this purpose. All the gauges should be connected to the nearest GTS Bench Mark and observations carried out as per standard practice.

(*f*) For working out a gauge-discharge curve and also to estimate maximum flood by slope-area method, the flood marks in the vicinity of the site should be recorded by local enquiries.

It would be useful to present the hydrological data availability for various years in the form of a histogram.

3.2.3. *Sediment Data*

For planning sediment excluding and/or ejecting devices at the headworks and in the canal system and to evolve a suitable gate regulation for satisfactory sediment exclusion, it is necessary to have data on the sediment load carried by the river for as such period as possible. It is required especially for the flood season when the sediment carried will be more. If no data on sediment is available, sediment observations should be started immediately along with gauge and discharge observations at the site to assess the quantity and quality of sediment carried in the river. The sediment sampling should be done as per standard practice. If the quantity of sediment brought by the river is excessive, the pond levels have to be fixed carefully taking the sediment data into account. This is especially important when the pondage is proposed to be provided to meet diurnal power fluctuations also.

3.2.4. *Evaporation Data*

Evaporation is an important factor to be considered while working out the water requirements of the crops and hence the diversion requirement. The available data on evaporation by pan measurements, etc. should be collected and analysed.

3.2.5. *Surface and Sub-surface Investigations including Laboratory Tests*

For proper design of the diversion structure and its components, it is very necessary to have a complete picture of the surface and sub-surface conditions. These are required both for deciding the type of structure to be provided and also its safety. The foundation may be of rock or alluvium and depending on the same, the investigations may be different. These are discussed below :

(*a*) Bore holes should be driven at specified intervals covering the entire area of the barrage or weir and its appurtenant structures, and the location of these bore holes should be indicatad clearly in the survey sheets. Correct log charts are to be prepared clearly. The borings should be generally carried out to hard rock level or to a depth of about 15 to 25 m below the deepest river bed level depending on the strata and type of structure. The bore holes should be at the rate of at least one in each bay proposed. If possible, two bore holes in each bay, one at the upstream cut off and another at the downstream cut off, should be obtained. When the presence of clay in the foundation is detected, these are, however, essential so that the extent, depth and location of the clay layer could be correctly assessed.

(*b*) If rocky strata is available at shallow depths, trial pits may be excavated to determine the depth of overburden, loose deposits, depth of weathered zone and extent of joints and fissures so that necessary function treatment for preventing excessive seepage loss, etc., can be worked out.

(*c*) Wherever the depth of overburden is large and comprises large size boulders and it is difficult to have ordinary boring methods, geophysical investigation needs to be carried out to locate the rock surface.

(*d*) Wherever impervious layers are encountered, the direction of its boundary *i.e.*, horizontal or sloping up towards the downstream or upstream should be ascertained. An impervious boundary sloping up on the downstream will cause constriction of seepage. This needs to be avoided by shifting the location of the barrage or weir slightly upstream.

(*e*) If the river bed consists of boulders or is made up of stiff soil, feasibility or otherwise of driving sheet piles to act as cut offs must be investigated properly.

(*f*) If piers or abutments are to be designed with reinforced cement concrete pile foundation, bearing capacity of the piles should be determined by driving test piles before finalising the design of the pile system, at sites where the bearing capacity of the soil is low at shallow depth.

(*g*) In sandy strata, standard dynamic and static penetration tests should be done below the position of each structure like abutments, piers, divide walls, centre of each bay, etc., for estimating the bearing pressures, likely settlement and necessity of settlement joints, etc. For boulder strata, plate bearing tests should be done. The depth of penetration may be at least half the bay width of the barrage or weir. The interval may be decided depending on the strata encountered.

(*h*) Wherever the main floor of the structure is to be designed as an R C.C. raft supported on an elastic medium, in situ tests for determining the modulus of sub-grade reaction at the proposed foundation level should be conducted. If there is a wide variation in the properties of the foundation material, the length of the structure should be split up into suitable sections isolated from each other by means of double piers and the modulus of sub-grade reaction must be determined at every section.

(*i*) Soil samples should be collected at suitable intervals of depth and their properties such as classification, unit weight, angle of internal friction, void ratio, specific gravity, grain size distribution, etc., should be determined by sieve analysis and other laboratory tests.

(*j*) Wherever clayey strata is encountered, undisturbed samples of the clay layer should be obtained, one from the proposed foundation level and another 5 m below the foundation level for each bay. These samples should be analysed to determine the cohesion, unconfined compressive strength, moisture content, dry density and sensitivity and consolidation characteristics.

(*k*) Wherever clayey strata is proposed to be treated by means of sand drains or stone columns, coefficients of permeabilities and consolidation should be determined in the vertical as well as radial directions.

(*l*) Observations of water table in the vicinity of the barrage or weir site should be carried out. The effect of higher pond levels leading to rise in water table with consequent water logging in the adjacent areas and the attendant problems thereof should also be investigated.

(*m*) Field permeability tests should be carried out to assess the extent of seepage losses from the pond and thereby the dewatering requirement.

3.2.6. *Diversion Requirements*

The quantum of water that is required is to be diverted through the barrage or weir has to be determined. For this purpose, detailed studies of the command area, cropping pattern, drainage, water requirements at the farmly outlets, any supplementary source of water, evaporation losses, percolation losses, requirements for power, drinking water supply, industrial uses, riparian rights etc., have to be made. The full supply level at the head regulator has to be worked out.

3.2.7. *Construction Material*

The type of structure which can be economically built at the site will depend on the availability of construction materials in the vicinity. Hence a detailed survey of, the materials like stone, limestone, bricks, sand, gravel suitable earth, etc., has to be made with regard to both quantity and quality of the same. The lead and lifts with necessary transport arrangements will also have to be found out. Laboratory and field tests should be carried out for determining the quality of aggregates and earth materials. For lime stone, its hydraulicity, strength and durability have to be assessed. It is preferable to prepare a map indicating the quarries, approach roads, colonies, etc., for a proper assess ment of the available construction materials. Comparative estimates of the type of structure will have to be made depending on the eassy and uninterrupted supply of the required quantity of quality materials.

3.2.8. *Communication System for Access to the Site*

Investigations will have to be carried out regarding the existing communication facilities like railways, roadways, navigation, telecommunications etc., at the site of work If some of the facilities are going to be dislocated due to the proposed work, their relocation will have to be thought of. Also additional facilities required both during construction and operation will have to be investigated and provided for.

3.2.9. *Other Miscellaneous Studies*

In addition to the investigations mentioned in the foregoing paras, it is necessary to make some more important studies which will have a bearing on the designs. Some of these are pond survey, fish pass, log chute, rail/road bridge across the barrage and data relating to ice problems. It is also necessary to make a survey of the construction colonies and labour camps which should be nearer to the site of work with easy accessibility.

3.2.9.1. *Pond Survey.* The upstream area which will be submerged by the normal pond and the area within the afflux bunds which will have to be acquired due to the construction of the barrage or weir will have to be surveyed. Details of the immovable properties coming within the same should be recorded and their values assessed before the works are started so that any disputed claims arising later could be settled without difficulty.

3.2.9.2. *Fish Pass.* In some of the barrages or weirs, arrangements for migratory fish have to be made. These are called fish passes either in the form of ladder or lock. The type, layout etc, of the arrangement need to be finalished in consultation with the concerned Fisheries Department. Necessary data in this respect should be collected.

3.2.9.3. *Log Chute.* In some of the barrages or weirs, there may be necessity of providing log chutes. To justify the necessity for the same, data regarding their numbers, sizes, periods of handling, cost of handling and other relevant data thereof should be collected and analysed.

3.2.9.4. *Rail/Road Bridge Across the Barrage.* Wherever communication facilities are to be extended from one bank to the other, it is usually and economically combined with a barrage. It may be a road bridge or a rail bridge or both combined. Necessary data regarding the same are to be collected for designing the structure and also the barrage itself over which the bridges are carried. In the case of a road bridge, data regarding the type of bridge, width of roadway, foot-paths, class of loading and other facilities lighting, pipe lines, cable ducts, hand rails etc., should be collected. In the case of a rail bridge, data regarding the gauge, line of traffic, etc. need to be collected in consultation with the railway authorities.

3.2.9.5. *Data Relating to Ice Problems.* There may be cases where barrages or weirs have to be constructed in high altitudes where the problem of ice formation exists. For proper safe design of the structure including the operation of gates where deicing facilities have to be provided, it is necessary to collect data regarding the ice thickness, maximum and minimum temperatures and rate of variation of temperature.

3.3. Environmental Considerations

In addition to the various considerations discussed in the foregoing paras, another important consideration on environment is gaining ground now. This has also to be rightly looked into. The planning, construction and operation of irrigation, hydroelectric/multipurpose projects have considerable impact on navigation, fish culture, wild life, recreational aspects and overall ecology of the affected regions. Some of these aspects on the ecology of the region as well as the overall environment are irreversible in nature. It is, therefore, necessary that a careful evaluation is made of these impacts, whether good or bad, before the project is undertaken. Necessary measures should be planned well in advance to mitigate, wherever feasible, the adverse impacts. The minimum surveys and investigations required on these are indicated below.

3.3.1. *Fish Culture*

This is already discussed under para 3.2.9.2. Some restrictions may also have to be imposed on fishing in the project areas. Owing to the diversion of water into the canals, there may be modification in flow patterns and quantums along the rivers/channels and their effects on fishing downstream should also be evaluated. Measures should be identified to reduce the quantity of the trash fish and at the same time increase the availability of quality fish in the affected areas.

3.3.2. *Wild Life Habitat*

A survey plan of the areas likely to come under the pond which could be constituting as encroachment on wild life (fauna) habitat as a result of the proposed structure should be made. This may not be applicable in the case of barrage/weir structure as in the case of storage dams. However, it would be worth investigating the special importance if any to wild life in their annual/seasonal migration. Necessary details regarding rehabilitation if any, and waterlogging problems which might affect wild life due to the pond should be worked out in consultation with the appropriate department of the State Government dealing with forest and wild life.

3.3.3. *Historical and Cultural Repercussions*

Sites of great historical and cultural importance should be carefully looked for during the investigations and where such sites cannot be avoided, a complete inventory of these should be made. The feasibility of shifting such monuments to safe areas nearby wherever this is feasible should also be investigated. Damages, if any, likely to be caused by the pond to the outstanding scenic beauty (flora) should also be ascertained.

3.3.4. *Other Ecological Factors*

Large hydraulic structures will result in modification in the natural flow patterns of the rivers. The regime of the river is likely to be disturbed. Problems likely to arise due to silting/scouring in the river bed, impact of flood problems, salinity of the flow in the river channel and other similar ecological factors should be thoroughly investigated.

Hydraulic structures provide the scope for augmenting manifold the water-based recreation facilities including sport fishing. Development of these should be investigated.

3.3.5. *Water Logging Problems*

Water-logging is the phenomenon by which the soil of a land is affected or damaged by too much water and due to poor drainage. This may be caused by too much of irrigation also. The inflow of water in the soil by infiltration from rivers causes water-logging around the headworks. Water-logging leads to many undesirable results such as breading places for mosquitoes, infertility of soil, subsidence of building nearby, etc. Hence this aspect has to be carefully considered while planning a diversion scheme.

CHAPTER 4

Location and Alignment

4.0. Introduction

For the successful functioning of the headworks, proper location and alignment of the barrage or weir are very important. Various considerations are involved in the selection of the site for location of the diversion structure and its alignment thereat. These are discussed in this chapter.

4.1. Location

The suitability of any site for locating the barrage or weir has to be analysed from the considerations for the components of the barrage or weir like undersluices spillway and the head regulators, each one having its own requirements to be satisfied for efficiency. An ideal site is one which satisfies the requirements of all the three items. But it is very difficult to find such a site in general. Hence a balance has to be struck after considering the merits and demerits of what are all available at the site.

4.1.1. After the commanded area has been selected, the location of the headworks has to be carefully selected such that besides being nearer to the service area, the height of the structure would also be reasonable. The combined cost of the construction of the headworks and the canals upto the commencement of the service area should be as small as is consistent with the efficiency of the works.

4.1.2. The nature of the soil through which the canals will have to run with a particular location of the headworks and the nature of the foundation materials at the location of the headworks itself will be considered in depth before deciding about the location of the headworks. The geology of the vicinity of the site is also another important factor. While fault zones, etc., should be avoided as far as possible, in case no other battle site could be found, the geological findings will indicate the foundation treatment or any appurtenant works needed to be adopted at that site. As has already been said, a site may have a number of advantages and disadvantages viewed on different considerations. The best site under the circumstances would be the one entailing the lowest cost keeping in view both the construction and maintenance charges.

4.1.3. The different items discussed below will serve as guidelines in selecting a satisfactory site for locating a barrage or a weir.

4.1.3.1. *Courses of the River.* The best site for locating a diversion structure is in a straight reach of the river, wherein the velocity of the flow should be fairly uniform and the sectional area of the river fairly constant. The banks should be firm, well defined and inerodible. A narrow gorge may reduce the length of the structure, but due to greater velocity and depth of flow, heavier sections may need to be provided. On the other hand, a wider river section may induce siltation due to reduced velocity besides necessitating construction of longer tie bonds.

However, from the consideration of silt entry into the canal, certain amount of curvature to the river course is desirable. Due to the curvature, helical flow is developed and the bottom layers of silt charged water is diverted away from the canal off-take on the outer side of the curve. When the canals take off on both the banks, the one on the inner side of the curve will draw more silt.

It has also to be seen that the course of the river has not changed for many years at the prospective site of the structure. In the case of shifting rivers, proper measures will need to be adopted to contain the river in the present course. The length of the fixed course of the river should be at least 0.4 times the meander length. When this much length is not available, the diversion structure may be located at the nodal point.

4.1.3.2. *Availability of Construction Materials.* The proposed site for locating the diversion structure should be in proximity to the quarries supplying construction materials and good communication facilities between the quarry site and the barrage/weir site should be ensured.

4.1.3.3. *Nature of the River Silt.* One of the factors that could influence the choice of the site is the nature of the river silt It is to be determined whether the silt has got fertilising properties and if so, upto what proportion the entry of the silt could be allowed in the head regulator and then down the canal in the case of irrigation project. This may govern the selection of the straight or curved reach for the location. The soil through which the canal runs should be able to withstand the velocity of flow which can carry the silt down the canal. This factor will also influence the location of the head regulator and thus the diversion structure.

4.1.3.4. *Slope of the River.* If some storage is required at the diversion structure, the structure may need to be located in a reach where the slope of the river is gentle. On the upstream side of the proposed location, if the river slope is steeper, then the discharging capacity of the diversion structure would be higher, but the structure will have to be designed to withstand higher velocities. Steeper slopes will restrict the length on the upstream upto which the effect of back water would be felt. Steeper slopes will mean shorter lengths of flood protective bunds and consequent reduction in cost.

However, immediately on the downstream of the proposed location, the slope of the river should be very flat for at least a short distance. This would provide a cushioning effect on the jet of flow down the crest of the structure, thus protecting the foundations from scour. Further downstream a steeper slope would be desirable so that the flow is drained off quickly and submergence of irrigable area is avoided.

4.1.3.5. *Condition of the Banks.* The banks at the proposed location should be firm and not erodible. The banks should be high so that the adjoining areas are not submerged during high floods. Otherwise costly flood protective embankments would be necessary. If high banks are not available, a reach where the high flood level is least above the natural ground level may be chosen subject to other aspects being satisfactory. After some length of the canals, the ground should have a steeper down slope so that the canal cutting which is generally heavy in the head reaches gets reduced.

4.1.3.6. *Width of the River Bed.* The width of the river at the proposed location should not be too wide which induces irregular silting forming shoals on the upstream and consequent irregular and likely cross flows. A very narrow width of the river also will reduce the waterway and induce high intensities of flow, which will necessitate deeper cut offs and costly foundations and protective works. Hence the width of the river should be slightly less than the normal width. The lengths of upstream afflux bunds and tie-bunds should be kept minimum so that there is no outglanking of the diversion structure and cost of training works is also minimum. Chapter 2 of Volume II—River Training Works may also be referred.

4.1.3.7. *Foundation Conditions.* If rocky strata at shallow depths is available, the foundation of the diversion structure can be had relatively at a lesser cost. Even if there are fissures in the rocks at shallow depths, proper foundation treatment by way of grouting, etc., can be done. If the foundation materials are made up of sand going to great depths, it presents no problems as the diversion structure can be successfully designed as per the principles discussed in other subsequent chapters. If impervious layers are encountered in between, proper precautions to release built up pressures in the substrata, if any, will need to be taken. On the upstream side, the cut offs could be embedded into impervious layer to reduce seepage pressure. On the downstream side such an embedment should be avoided so that pressures are not locked up If it is not possible, proper drainage arrangements should be provided.

4.1.3.8. *Confluence of Tributaries* The location of the diversion structure should generally be below the confluence of the main tributary of the river. However, wherever it becomes necessary to locate the structure upstream of the confluence due to various other considerations, the structure should be located as far above the confluence as possible to reduce the cost of cross drainage works. The location of the structure in the main river with a tributary flowing parallel to the main river should be avoided as there would be possibilities at a later date of the main river breaking into the tributary and outflanking the structure, thereby rendering it infructuous.

4.1.3.9. *Quantity of Floating Debris.* Location of the diversion structure at reaches where the quantity of floating debris carried by the river is large should be as far as possible avoided. Otherwise preventive measures to safely remove them from entering into the main canal and to pass them down the river will have to be suitably incorporated in the structure.

4.1.3.10. *Suitability of Site for Undersluice.* If in a diversion structure, there has to be provision for undersluice, a site where there is a deep channel of the river on the side where the canal has to take off, may be selected. In the case of canals on both the banks a site with deep channels on both sides and a third in the middle could be a better location.

4.1.3.11. *Head Reaches of the Canal.* The head-works has to be located generally nearer to the service area such that the idle length of canal before it starts serving the area is minimum. However, the advantage of the site has to be considered from the cost aspect also since in such cases the pond level may have to be kept higher. So in these circumstances the extra cost due to the idle length of the canal by locating the diversion structure at a higher site has to be weighed against the cost of the structure at a lower site.

4.2. Alignment

After having selected a suitable site for locating the diversion structure from various considerations discussed under para 4.1, and its sub-paras, the alignment of the structure at that site has to be fixed.

4.2.1. *Alignment with Reference to the Course of the River*

The alignment can be fixed in two ways with respect to the direction of the current, viz , (*i*) at right angles to the course of the current and (*ii*) oblique to the current. Each one has got its own advantages and disadvantages.

4.2.1.1. *Alignment at Right Angles to the Course of the River.* Generally the diversion structures are aligned at right angles to the general flow of the river. (See Figure 4.1). The advantages over such an alignment are :

(*a*) The unit discharging capacity would be maximum.

(*b*) The flow would be more or less uniform over the length of structure.

(*c*) It is most suited for sandy and silty foundations.

(*d*) It is more economical and practicable.

4.2.1.2. *Alignment oblique to the Currents.* Oblique alignment of diversion structure to the current is avoided as far as possible. In case, this cannot be avoided due to certain other pressing reasons, the river training has to be done so as to provide uniform flow across the structure and attract the channel to the bank where canal takes off.

Alignment of the diversion dam oblique to the current could be arranged in two ways by having the splay either towards the canal side or away from it. (See Figure 4·1). In the first case, a deeper channel is maintained along the face of the head regulator of the Canal. In the second case, the main current is deflected away from the Canal which is a desirable feature where boulders and heavy gravel are carried by the river. The difficulty with both the cases is that if the river course gets altered at a later date, it would be difficult to remedy the defects due to the oblique alignment.

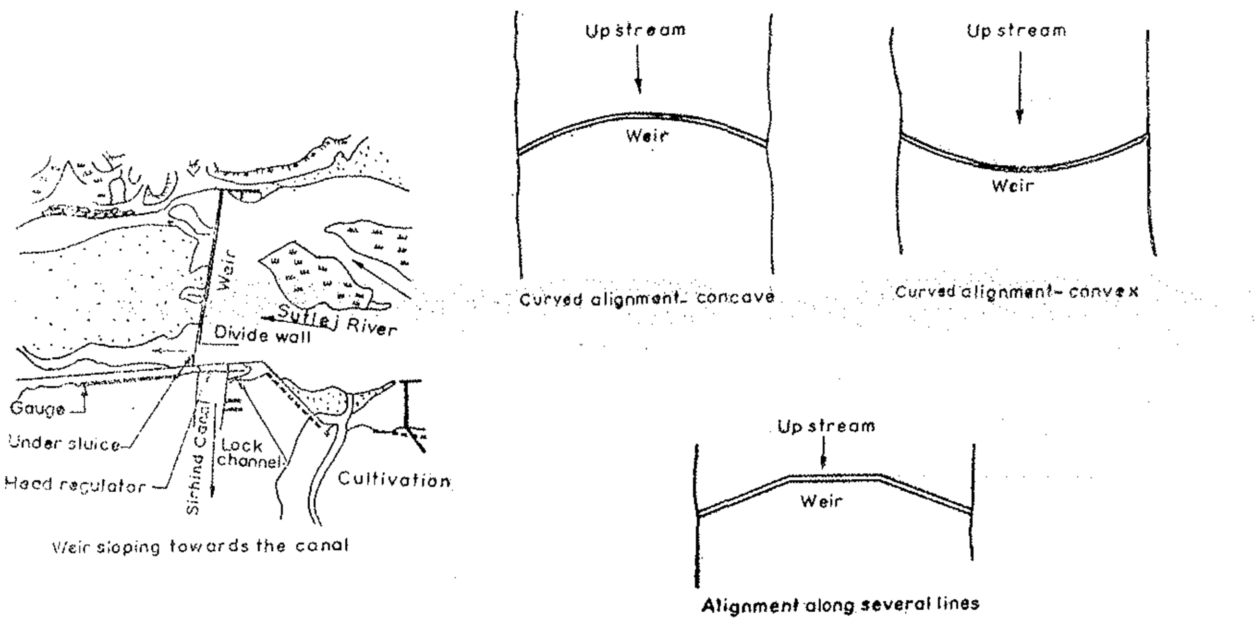


FIGURE 4.1 : Types of Alignment.

4.2.2. *Alignment with Reference to Geometric Shape*

The alignment of the diversion structure can be arranged in three ways, viz. (*i*) straight along a single line (*ii*) Curved and (*iii*) straight or curved along two or more lines.

4.2.2.1. *Straight Alignment in Single Line.* This is the alignment generally followed in many of the diversion structures. The flow lines on the downstream of the structure would be parallel to the banks.

4.2.2.2. *Curved Alignment.* The curved alignment of the diversion structure can be had with concavity upstream or convexity upstream, both as viewed from the downstream side. (See Figure 4.1). The alignment with concavity upstream is suitable when the location of the structure is proposed at a point where the river after having widened is narrowing down to normal course. The flow would converge to the centre.

The alignment with convexity upstream is not recommended as it diverges the current towards the banks exposing them to scour. Moreover, this type of alignment is also weak structurally.

4 2.2.3. *Polygonal Alignment.* This type of alignment is only a combination and modified version of the other two alignments. In this case also concavity upstream is preferable. (See Figure 4.1).

4.3. Hydraulic Model Studies

While some of the minor diversion structures can be located and aligned at a suitable site after considering the merits and demerits of some selected sites and also by experience, in the case of major diversion structures, recourse is generally taken to select a good and exact location in the vicinity of some selected reach and also to determine the satisfactory alignment at that particular location through the help of 3-D hydraulic model studies. Further details on the model studies are discussed in Chapter 7 of Volume II—Model Tests.

4 4. Corrective Measures

It is always very difficult to select an ideal site for the location of the diversion structure and its regulators satisfying all the requirements discussed in the earlier paras. Hence it often becomes necessary to select a site satisfying most of the requirements and for the rest, some corrective measures will have to be incorporated in the layout and designs. These include properly designed guide bunds, spurs, flood protective embankments, pilot channels, silt excluding devices, etc. These have been discussed elsewhere in this manual and reference may be made to these.

CHAPTER 5

Planning and Layout

5.0. Introduction

Having selected a site for locating the diversion structure, be it a barrage or a weir, planning has to be systematically done to decide upon the various important parameters of the structure which will govern the designs. The layout of the structure can be finalised with reference to all the requirements which the diversion structure has to cater for. The parameters have to be decided on economic and general considerations. These are discussed in this chapter and some illustrative figures of some important barrages are also incorporated.

5.1. Planning

The requirements, for which the diversion structure is required to be constructed, should be listed out and the type of structure selected. This will help in planning for the various components and appurtenant works to be incorporated and evolving operating conditions. The parameters for which the barrage or weir has to be designed are generally design floods, pond level, afflux, waterway, crest levels, free board, type of cut off to be provided, etc. While planning the structure, the river diversion arrangements during construction should also be planned in advance. The planning aspect does not end with deciding the parameters alone. It has to go side by side with the construction, procuring materials, etc. Planning the construction programme has an important bearing on the layout, especially in the provision of joints between blocks of the structure.

5.1.1. *Design Flood*

The diversion structure has to be designed providing sufficient waterway to pass a flood with certain return period safely. The free board of the structure as also the foundations have to be designed for a higher flood with certain return period. Usually, in the case of diversion structure of minor importance, the waterway is designed to pass a flood of 1 in 50—100 years frequency and free board is provided for a flood with 500 years frequency. For more important structures, the waterway is designed to pass safely a flood of 100—200 years frequency and free board is provided for 500 years flood. Hence the design flood to be adopted for the structure has to be carefully decided upon. Gauge-Discharge curve upto these flood values at the site should also be worked out by using statistical methods by fitting proper equations to the observed Gauge-Discharge Curve. For various methods of preparation of Gauge-Discharge curve, IS : 2914-1964—Recommedations for estimation of discharges by establishing stage-discharge relation in open channels. See Figure 5.1 for a typical Gauge-Discharge Curve.

5.1.2. *Afflux*

As a result of the construction of the diversion structure across the river, there will be a rise in the flood level on the upstream of the structure and it is called the afflux. (Though it is confined in the beginning to a short reach of the river it extends gradually very far up till the final slope of the river upstream of the structure is very much the same as it was before the construction of the structure.) While higher afflux can be permitted in steep reaches of the river with bouldery or rocky bed, due to proximity of the structure to important populated or industrial towns in the alluvial reaches, permissible afflux will be limited in many cases. The amount of afflux will determine the top levels of guide bunds and marginal bunds, their sections and length and all the top levels of the components of the structures like abutments, piers, flankwalls, etc. Afflux will also govern the dynamic action downstream of the work as well as the depth and location of the hydraulic jump. By providing a higher afflux, the waterway can be reduced but the cost of training works will go up and the risk of failure by out flanking will increase. Hence, it is very important that the premissible afflux for the design flood is decided upon, so that designs can be carried out accordingly.

5.1.3. *Free Board*

Once the permissible afflux is decided, the necessary waterway can be accordingly worked out and upstream water level estimated for the design flood. Over the high flood level obtained from the Gauge-Discharge Curve on the downstream side

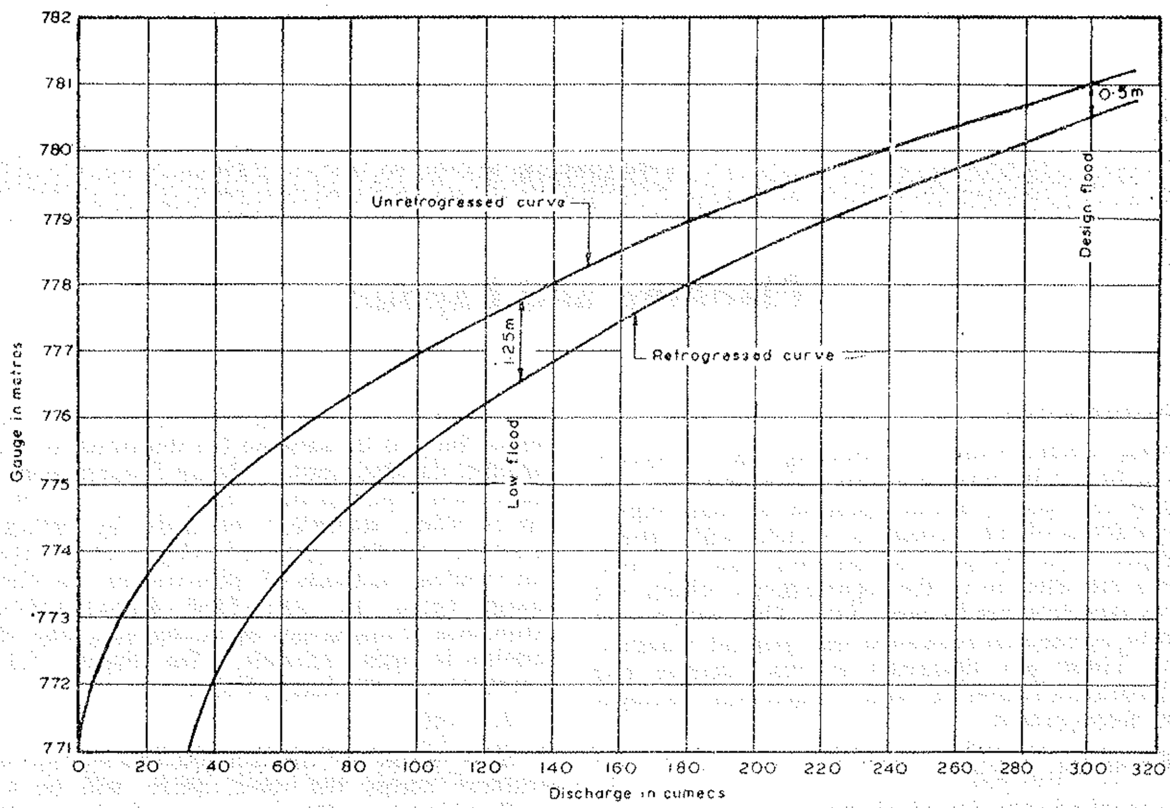


FIGURE 5.1. : Typical Gauge Discharge Curve.

and estimated on the upstream side, sufficient free board has to be provided so that there is no overtopping of the components like abutments, piers, flank walls, guide bunds, afflux bunds, etc. The free board to be provided depends on the importance of the structure. Generally 1.5 to 2 m free board above the affluxed water level on the upstream side and above the high flood level (unretrogressed) on the downstream side are provided. While deciding about the free-board, the likelihood of accretion of the upstream bed of the river due to excessive silt content of the river should also be considered. It will be relevant to point out damages have occurred in quite a few structures due to inadequate provision of free board. The free board generally takes care of wave splash, higher afflux caused due to faulty gate operation, variation in the coefficient of discharge assumed for calculations and the flood value itself. The free board is provided over the affluxed water level due to a flood with 500 year frequency. However, if the pond level is higher than the water level for 500 year flood, the free board will be provided over the pond level. Hence, it is important to decide about the allowable free board for the safety of the structure.

5.1.4. *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 pond level has to be carefully planned so that the required water can be drawn without difficulty. Adding the losses through the head regulator, and the working head to drive the required discharge through the head regulator, the pond level can be fixed. Sometimes the pond level has to be fixed allowing for the diurnal variation in the water level due to release from any hydel stations upstream. If the river carries lot of silt, then there is a likelihood of encroachment into the pond level after some years. Hence to cater for the same, there might have to be a 'future' pond level also fixed in some cases as in Kosi Project. The 'future' pond level can also be fixed for any possible increase in the service area at a later date. The structure will have to be designed to be safe against the increased pond level also. The provision of a high pond with a level almost equal to High flood level or above the same has to be planned very carefully since such a provision is likely to induce shoaling upstream. In those cases, the operation of the gates will have to be efficiently formulated and the silt excluding devices have to be suitably designed. As the designs of the rafts, piers, abutments, etc. are governed in many cases by the pond level condition discussed separately in the chapters on designs, fixing of a suitable pond level from various considerations is quite important.

5.1.5. *Waterway*

The limit placed on the afflux also limits the minimum waterway. A likely figure to adopt for the width of waterway would be the minimum stable perimeter of Lacey for the highest flood discharge. In the case of bouldery reach, the width available at the site is limited by firm banks. It is generally thought that by limiting the waterway, the shoal formation upstream can be eliminated. However, it increases the intensity of discharge and consequently the section of the structure becomes heavier and cost increases though the length of the structure is reduced. There is a greater risk of outflanking and damages due to local concentration of flow. Many of the existing structures have a clear waterway from 10 to 25 percent more than Lacey's regime perimeter. The ratio of the waterway between abutments to the Lacey's regime perimeter is known as the Looseness factor. Generally the looseness factor varies from 1.1 to 1.5 though there are a few cases with looseness factor less than 1, which are more prevalent in bouldery reaches. Table 5.1 gives a comparative statement of the various parameters adopted in the case of different barrages in the country. Before designing any structure, the value of looseness factor which has to be adopted for the structure has to be decided upon. This has to be arrived at after a careful review of the permissible afflux, intensity of flow, river bed and bank conditions, width available between banks at high flood level, dewatering problems, etc. It may also be noted that in the case of limited width of the river, and attempt to increase the same by excavation and suitable approaches and exit will have to be done after extensive model studies since some of the additional waterway thus created may become dormant after some time.

5.1.5.1 *Looseness Factor* : As already stated in para 5.1.5, the values of looseness factor for a number of barrages designed and constructed over a few decades vary from 1.1 to 1.5. From the performance of these barrages, a feeling arises that with high looseness factor more than even 1, there is a tendency for shoal formation in front of the barrages which causes damages and maintenance problems. Hence, some of the designers tend to go in for as low a looseness factor as possible, even to the extent of 0.5. But fixation of such arbitrary values of looseness factor should not be allowed. A need for certain recommendations in this regard has been felt.

5.1.5.1.1. It may be seen from the discussions in the various preceding paras that the fixation of the waterway of a barrage is closely linked with various parameters such as discharge, cross-section of the river at the axis, nature of river bed materials, silt content of the river, crest levels of undersluices, river sluices (if any) and spillway bays, permissible afflux, permissible intensities of discharge, dewatering problems, economy, etc. The prototype performance data of many barrages are also not readily available. Hence, it would be really not desirable to recommend any particular value of looseness factor to be adopted unless a detailed study of all these parameters is carried out in a scientific manner, preferably in a Research Station. After the results of such studies are available and certain conclusions and recommendations are drawn up by experts in the discipline, the issue regarding the recommendation of any optimum range of looseness factor with reference to discharge and river bed material could be reviewed.

5.1.5.1.2 However, till such time, perhaps the looseness factor may be adopted to be near about the value determined on the basis of dominant flood instead of design flood. The available cross-section of the river also should be taken into account. It is felt that in any case, the value of looseness factor should not be lower than about 0·6 (on the basis of design flood). Wherever possible, hydraulic model tests should be carried to study the performance with the proposed looseness factor.

5.1.6. *Crest Levels*

Waterway to be provided, crest levels of undersluice and spillway and afflux permitted are all inter-linked. A suitable combination has to be evolved keeping in view the limits for each. The crest level of the undersluice is generally kept at average bed level or slightly raised by a meter or so depending on the nature of silt materials brought down the river and silt exclusion devices proposed. The crest level of the spillway is generally kept about 1 to 2 m above the crest level of the undersluice. However, it has to be noted that the crest levels have to be fixed finally satisfying the conditions on afflux and waterway.

5.1.7. *Type of Cut off*

Another important aspect to be decided is about the type of cut off to be provided. Cut off may be of reinforced concrete or sheet pile or wells depending on the nature of foundation strata. As will be seen from the chapter on Hydraulic Designs, upstream cut off is required for safety against scour and also to reduce the seepage pressures beneath the floor. Downstream cut off is required for safety against scour and also safety against piping action. In alluvial strata, generally sheet pile cut offs are provided due to the case of construction. In bouldery beds, reinforced concrete cut offs are provided as it is difficult to drive sheet pile cut offs. By planning to have the type of cut off after studying the bed strata, the sections of the structure required could accordingly be worked out. It will also facilitate procurement of necessary materials for the cut offs.

5.1.8. *Construction Programme*

The construction programme has to be planned taking all the relevant aspects like available working season, machinery, labour, materials, target, etc., into consideration. This has got a bearing on the provision of expansion joints between blocks of bays. Generally,

TABLE 5.1

Comparative Statement of Various Parameters of Diversion Structures.

	Parameters	Rupar (Sutlej)	Merala (Chenab)	Sulmanke (Sutlej)	Islam (Sutlej)	Ferozepur (Sutlej)	Sukkur (Indus)	Panjnad (Chenab)	Sarda Barrage (Sarda)
		1	2	3	4	5	6	7	8
	Year of Completion	1885	1907	1922	1922	1924	1925	1927	1928
1.	Design Max. Flood Discharge in cumecs	8923·5	20340	9207	7790	12748	42493	19830	16997
2.	Silt Factor	1·00	1·00	1·00	1·00	1·00	1·00	—	3·00
3.	No of Spillway Bays	—	—	—	16·00	20·00	54	43	30
	Undersluice	12	—	—	8	9	12	4	4
4.	Width of each Bay in metre Spillway	—	—	18·3 & 9·15	18·3	18·3	18·3	18·3	15·24
	U/Sluice	6·1	—	—	18·3	18·3	18·3	18·3	15·24
5.	Looseness Factor	1·70	1·98	147	1·15	1·10	1·45	1·56	0 95
6.	Afflux in metres	1·37	0·92	0·92	0·61	0·92	0·31	0·92	—
7.	Intensity of Discharge in Spillway cumec/m	10·22	14·87	13·57	19·33	13·76	35·32	23·05	39·03
	U/Sluice	21	33·18	22·31	31·78	24·07	35·32	23·05	39·03
8.	Thickness of Piers in Metres	—	—	2·13	—	2·13	—	—	2·44
	Glacis Upstream Slope Spillway	—	5 : 1	5 : 1	—	3 : 1	3 : 1	4 : 1	2 : 1
	Downstream	—	9 : 1	5 : 1	—	3 : 1	3 : 1	5 : 1	2 : 1
9.	Upstream U/Sluice	—	—	—	—	2 : 1	5 : 1	—	2 : 1
	Downstream	—	—	—	—	2 : 1	5 : 1	—	2 : 1
	Scour Upstream Depth Spillway	1·79 R	1·73 R	1·12 R	0·90 R	1·00 R	1·23 R	1·36 R	—
	U/Sluice	3·40 R	3·39 R	0·64 R	0·74 R	1·00 R	0·60 R	1·36 R	—
10.	Downstream Spillway	0·74 R	1·70 R	1·57 R	1·17 R	1·45 R	2·82 R	1·56 R	—
	U/Sluice	3·74 R	1·26 R	1·55 R	0·96 R	1·45 R	3·64 R	2·63 R	—
11.	Cut/off Spillway	—	8·02	7·32	9·15	4·57	—	6·71	—
	Sheetpile Depth in metres U/Sluice	—	4·88	8·23	9·15	4·57	U/S 5·49 U/S 5·49	7·01	—
12.	Launching U/S	—	2 : 1	2 : 1	2 : 1	2 : 1	2 : 1	2 : 1	—
	Slope D/S	—	2 : 1	2 : 1	2 : 1	2 : 1	2 : 1	2 : 1	—

TABLE 5.1 (*Contd.*)

Rasul (Jhelum)	Khanki (Chenab)	Trimmu (Chenab)	Kalabagh (Indus)	Gude Barrage	Nangal Barrage (Sutlej)	Harike (Sutlej)	Durgapur Barrage
9	10	11	12	13	14	15	16
1929	1935	1937	1939	1946	1948	1958	1956
24788	21246	18272	26912	31161·5	9207	18414	15581
1·13	0·97	1·00	1·00	—	—	1·00	1·00
—	48	—	42	—	—	22	24
—	—	14	14	—	—	9	10
3·66	6·1	—	18·3	—	—	18·3	18·3
—	—	9·15	18·3	—	—	18·3	18·3
1·76	1·91	1·41	1·46	—	—	0·97	1·145
0·21	0·92	0·92	0·92	—	1·52	0·92	0·92
17·94	13·48	21	25·09	27·51	37·92	33	21·1
40·24	35·97	31·6	30·67	—	—	33	25·26
—	—	—	2·13	—	—	2·13	2·13
—	9 : 1 (OPP)	5·67 : 1	4 : 1	—	—	3 : 1	4 : 1
—	15 : 1	5·67 : 1	4 : 1	—	—	3 : 1	4 : 1
Level	—	3 : 1	3 : 1	—	—	3 : 1	4 : 1
—	—	3 : 1	3 : 1	2 : 1	—	3 : 1	4 : 1
—	1·36 R	1·25 R	1 : 50 R	—	—	1·5 R	1 R
1 47 R	1·15 R	1·25 R	1·50 R	—	—	1·5 R	1 R
0·87 R	2·60 R	1·5 R	2·0 R	—	—	2·0 R	1 R
0·62 R	2·28 R	1·5 R	2·0 R	—	—	2·0 R	1 R
6.1	U/S 5·79	U/S 7·32	U/S 2·44 D/S 2·13	U/S 6·71 D/S 6 71	U/S 4·47 D/S 6·1	4·57	8·54
—	D/S 6·4	D/S 6·4	U/S 2·44 D/S 2·29	—	—	6·1	8·54
—	—	3 : 1	2·5 : 1	2 : 1	—	—	3 : 1
—	—	3 : 1	2·5 : 1	—	—	—	3 : 1

TABLE 5·1 (*Contd.*)

Krishna Barrage (Krishna)	Yamuna Barrage (U.P.)	Nanaksagar Barrage (U.P.)	Khara Barrage (U.P.)	Kota Barrage (Chambal)	Dakpathar (Yamuna)	Kosi Barrage (Kosi)	Sone Barrage (Sone)
17	18	19	20	21	22	23	24
1957	1959	1959	1960	1960	1961	1963	1965/67
33711	11615	1303	21246·5	21246·5	14448	26912	40510
—	—	—	3·00	—	4·0	1 : 3	1 : 4
—	22	7	34	19	22	46	60
70	4	—	5	—	4	10	9
—	—	6·1	18·3	12·2	18·3	18·3	18.3
12·2	—	—	18·3	—	18·3	18·3	18·3
—	—	—	1·15	—	1·2	1·45	1·45
—	1·22	1·22	0·92	0·92	0·92	—	1·22
—	29	—	27·23	—	35·32	25·84	31·6
—	34·85	—	47·4	—	42·75	34·85	35·32
2·44	2·44	2·44	—	3·2	—	2·13	2·13
—	3·2 : 1	3 : 1	3 : 1	3 : 1	Nil	3 : 1	4 : 1
—	3·2 : 1	1·25 : 1	3 : 1	3 : 1	3 : 1	3 : 1	4 : 1
—	3·5 : 1	2 : 1	3 : 1	3 : 1	3 : 1	3 : 1	3 : 1
—	3·5 : 1	2·5 : 1	3 : 1	3 : 1	3 : 1	3 : 1	3 : 1
—	—	—	1·25 R	—	1·25 R	1·25 R	1·5 R
—	—	—	2·00 R	—	1·25 R	1·25 R	1·5 R
—	—	—	1·25 R	—	1·25 R	1·25 R	2·0 R
—	—	—	2·00 R	—	1·25 R	1·25 R	2·0 R
5·18	—	—	U/S 3·05 D/S 5·49	—	—	U/S 8·84 D/S 9·15	U/S 6·4 D/S 7·47
—	—	—	U/S 3·96 D/S 4·57	—	—	U/S 8·84 D/S 9·15	U/S 6·4 D/S 7·32
—	—	—	—	—	—	2·5 : 1	2 : 1
—	—	—	—	—	—	2·5 : 1	2 : 1

TABLE 5·1 (*Contd.*)

Yamuna Barrage (Yamuna) Delhi	Jamuna Weir (Assam)	Gandak Barrage (Gandak)	Mukeriam Hydel Project Weir	Malan Barrage	Ramganga Weir (Ramganga)	Bharib Bank Barrage (West Bengal)	Sitabali Barrage (West Bengal)
25	26	27	28	29	30	31	32
1967/68	1968	—	—	—	—	—	—
8499	3133	24079	16997	850	5949	8074	1190
1·0	2·50	3·00	—	—	—	—	—
22	18	18	60	—	3	6	7
10	7	12 R/S 6	—	17	4	—	—
18·3	12·2	18·3	—	14·57	18·3	—	—
18·3	6·1	18·3	—	—	—	—	2·44
1·24	—	0·98	1·95	—	1·28	1·00	—
0·08	0·34	0·67	0·92	—	0·92	1·22	1·22
15·15	10·97	32·53	—	—	—	15·33	—
17	—	42·1	—	—	22·3	—	—
2·13	—	2·13	2·13	—	—	—	—
3 : 1	1·0	3 : 1	2 : 1	—	—	—	—
3 : 1	3 : 1	3 : 1	2 : 1	—	—	—	—
3 : 1	3 : 1	3 : 1	3 : 1	—	—	—	—
3 : 1	3 : 1	3 : 1	3 : 1	—	—	—	—
1·25 R	5	1·25 R	—	4	—	1·25 R	1·25 R
1·25 R	—	1·25 R	—	—	—	1·25 R	1·25 R
1·25 R	—	1·25 R	—	—	—	1·25 R	1·25 R
1·25 R	—	1·25 R	—	—	—	1·25 R	1·25 R
6·1	U/S 4·42 D/S 5·18	U/S 4·27 D/S 6·4	—	—	—	—	2·44
7·32	U/S 5·95 D/S 6·71	U/S 7·01 D/S 7·32	—	U/S 6·71 D/S 12·35	—	—	—
2 : 1	3 : 1	2 : 1	—	—	—	—	—
2 : 1	2 : 1	2 : 1	—	—	—	—	—

the barrage is constructed in units/blocks of 8 to 16 bays separated by double piers with expansion joints. This also depends on the nature of foundation strata. Once the units/blocks are planned, design of the barrage floor can be taken up accordingly.

5.1.9. *River Diversion during Construction*

The strategy proposed to be adopted for river diversion during construction also plays an important role in the design of the structures. For example, it may be decided to close the river in parts and pass the floods through already constructed portions. Due to limitations on the afflux, it may be necessary to have a lower crest level of certain bays which can be later raised to the final designed level. The designs will have to take care of such modifications. Hence, while planning the structure, aspects on river diversion also has to be considered carefully.

5.2. Layout

Layout of the diversion structure at the site and the alignment selected outlines the arrangement of the various components of the diversion structure such that the hydraulic performance of the structure is quite satisfactory under the given conditions. The various components of a diversion structure are spillway, Undersluice, River-sluice, Cut-off, Pier, Divide Wall, Abutment, Flank wall, Return wall, Guide bund, Silt excluding devices, Afflux bund, Approach embankment, Regulating arrangement, Fish pass, Navigation lock, Log chute, Bed protection, Head Regulator, etc. It is not necessary that all these components must be present in a diversion structure. Fish pass, Navigation lock, Log chute, Afflux bund, River-sluice and Return wall are some of the components which are to be added only if there is a necessity for one or more of these. While the arrangement of these components for small structures can be made by experience, the layout in major structures are generally finalised with the help of hydraulic model studies. It is important to know about the functions of the various components so that while deciding the layout it will be easier. These are discussed in the following paras.

5.2.1. *Spillway*

This is the main body of the diversion structure over which gates are installed for controlling the discharges and to raise the water level to the desired value to feed the canals. In the case of a barrage, it is a slender structure with upstream floor, upstream glacis, crest, down stream glacis, cistern (also called stilling pool) and end sill portions and cut offs at either ends of the floor. In the case of a weir, all these parts would be present except that the creast portion is raised upto the desired pond level) (also called operating level) or slightly below (due to provision of falling shutters if any) and it is massive. Typical sections are given in Figures 5.2 and 5.3. The length of the spillway bay (along axis of the barrage) depends on so many factors such as the size of the gate that can be manufactured, the cost aspects, etc. In general, the size can be anything between 8 m to 18 m. The width of the spillway section (along the flow direction) and levels of the profile will be determined from considerations discussed in the chapter on Hydraulic Designs.

5.2.1.1. As for the layout, the crest level can be fixed at average bed level or above, depending on the considerations of afflux, etc. However, it should not be kept more than 2 m (1·5 m preferred maximum value) above the crest level of undersluice or River-sluice.

5.2.3. *Undersluice*

Undersluices are a set of bays provided at the canal end of the diversion structure to keep the river under control aiming at the following :

(*i*) Maintain a clear and well defined river channel towards the head regulator.

(*ii*) to enable the canal to draw silt free water from surface only as much as possible.

(*iii*) scour the silt deposited in front of the head regulator.

(*iv*) in case of weirs fitted with falling shutters, to pass the maximum non-monsoon flood without necessitating the dropping of weir shutters.

5.2.2.1. The undersluice bays may be provided on only one flank or on both the flanks of the river depending on the number of canals taking off.

5.2.2.2. The layout of the undersluice bays should be thought of on the following consideration :

(*i*) the dimensions and level of undersluices should be such that there is the most effective exclusion of silt entry into the canal.

(*ii*) the dimensions are conducive to maintaining a clear channel towards the canal,

(*iii*) the capacity of undersluices is also influenced by diversion requirements during construction or of fair weather escapage.

5.2.2.3. The dimensions of undersluices are best determined by the help of hydraulic model studies. However, a few general guidelines for fixing the dimensions of undersluices are as given below :

(*a*) For ensuring proper scouring capacity the discharging capacity of the undersluice bays at pond level should not be less than twice the capacity of the canal near which they are provided.

(*b*) While a hard and fast rule cannot be laid down regarding the percentage of discharge to be

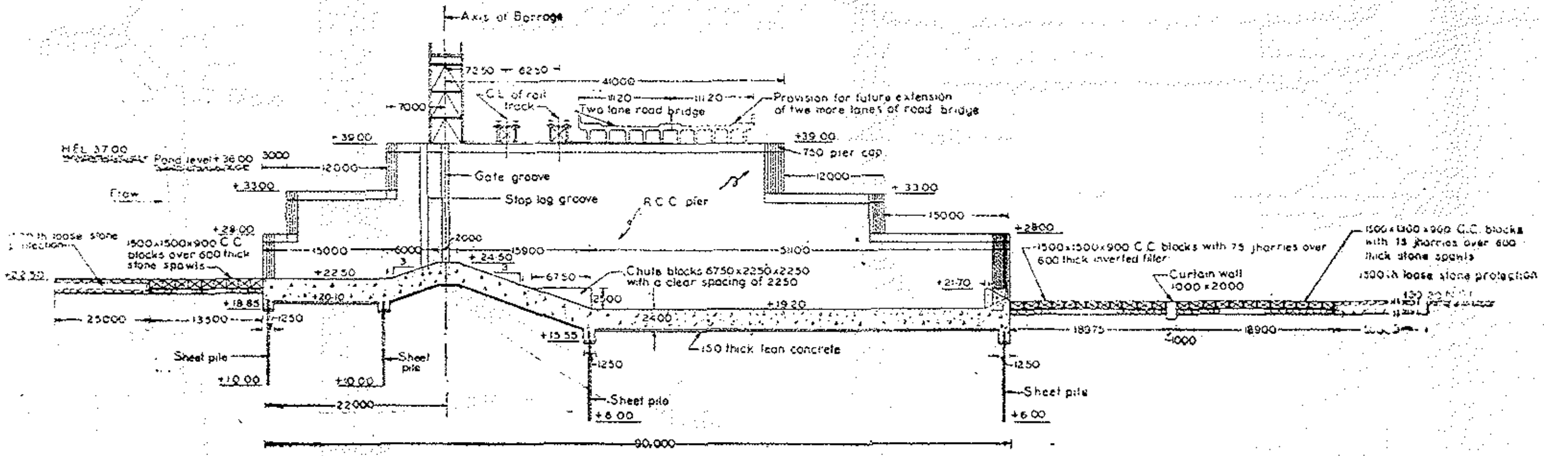


FIGURE 5.2 : Typical Section of Spillway of Barrage.

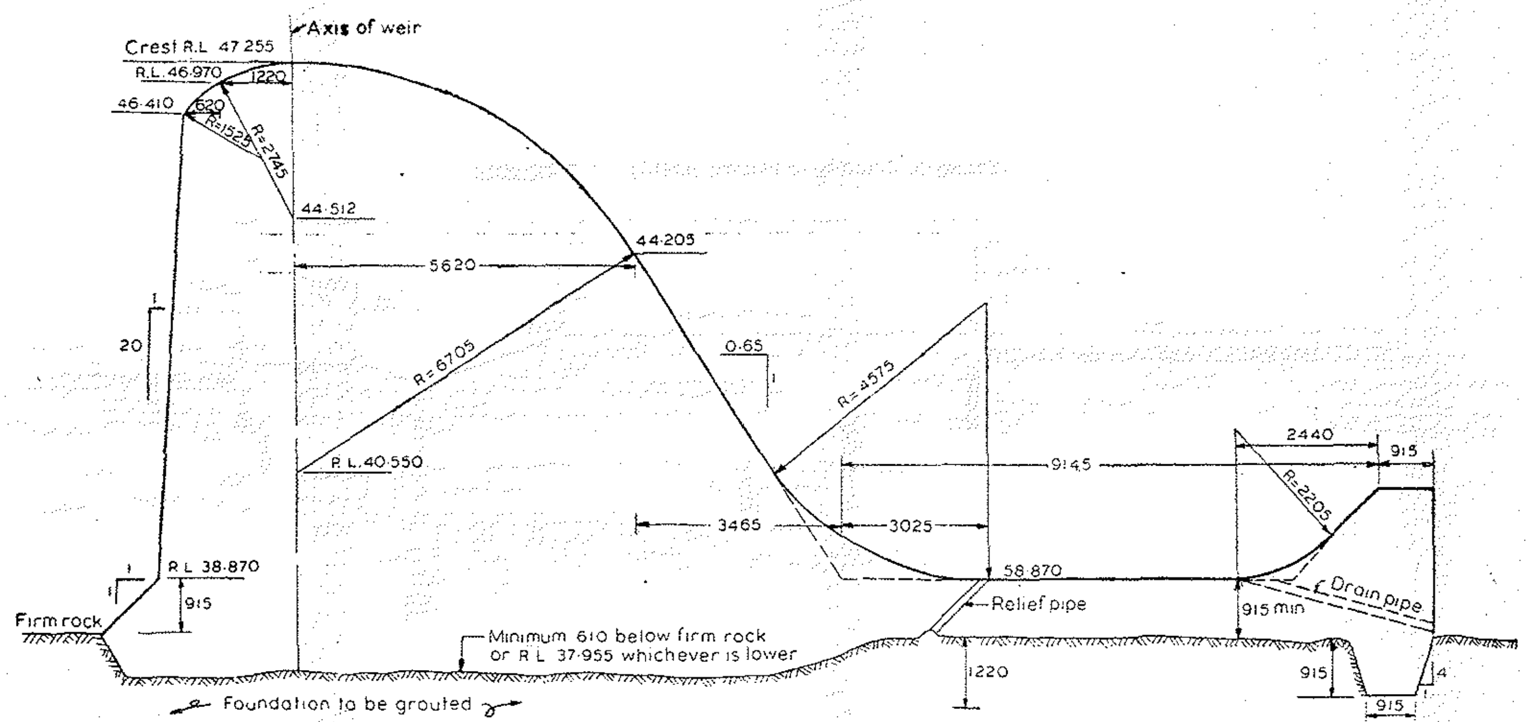


FIGURE 5.3 : Typical Section of Spillway of Weir.

passed through the undersluices, usually the capacity could be of the order of 10 to 20 per-cent of the maximum flood.

(c) The width of undersluice should be greater than the width of canal regulator and very often it is near about 1.5 times the width of the head regulator.

These are only general guide lines and sometimes there may be exceptions due to various parameters including the configuration and flow pattern of the river.

5.2.2.4. The crest level of the undersluice is generally kept as low as possible at the average bed level. The other levels of the undersluice profile will be fixed as discussed in the chapter of Hydraulic Designs. The bay length can be the same as that of spillway. Typical section of undersluice is shown in Figures 5.4 and 5.5.

5.2.3. *River-sluice*

River sluices are a set of sluices similar to the undersluices located in between the undersluice and spillway bays and separated from them by means of divide walls. These are generally provided in long structures for simplifying the operation of gates during normal floods and to have better control on the river. The section of rive-sluice bay will in general be similar to that of undersluice bay without silt excluder tunnels.

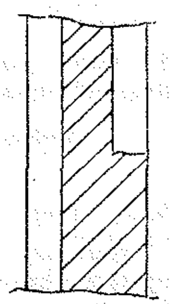
5.2.4. *Cut off*

Cut offs are barriers provided below the floor of the structure both at the upstream and downstream ends. This can be in the form of a concrete lug or sheet-pile or wells. They extend from bank to bank and also in the longitudinal direction along the flow. Their main purpose is to lengthen the seepage path below the structure and also to prevent piping action below the floor. The cut-offs in the longitudinal direction, also called cross-cut offs provide boxing effect, adding to the stability of the different units/blocks of the structure. They should be continuous and leak proof. The depth to which these are taken down below the floor levels depends on the safety factor adopted in the designs which is discussed in the chapter on Hydraulic Designs.

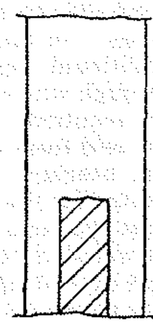
5.2.4.1. Where sheet piles cannot be driven, it becomes necessary to provide concrete cut offs. It is called diaphragm walling also wherein vertical continuous walls are cast-in-situ in a trench excavation. The wall/cut off is designed and constructed as a series of penals, the lengths of which usually range between 4.5 m and 7.5 m. Standard widths are 500, 600, 75(900 and 1000 mm.

5.2.4.1.1. In all cases, a trench to the required depth is first excavated by a specially developed grab, operated from a standard crane by means of a purpose-designed attachment including a Kelly bar and Kelly bar guide. The use of the Kelly bar ensures the correct alignment and verticality of construction. During excavation, the trench is kept filled with bentonite suspension which stabilises the trench walls, preventing caving in. Bentonite suspension is mixed on site and poured into the excavation as digging proceeds. The guide walls usually 0·6—0·9 m deep or more depending on ground conditions are constructed before excavation begins and also serve as top retaining walls or trench collars. Reinforcement is made up on site and formed into 3-dimensional cages for each panel. The case is positively located in the trench, spacers being used to ensure accurate centering and distance blocks fixed to ensure adequate concrete cover. Since the film of bentonite on the steel is removed by mechanical contact with the concrete, the reinforcement is completely encased by the concrete. Before concreting takes place, a steel stop end pipe is placed in position at one end of the trench and this acts as a form of shutter. In certain cases, pipes are placed at both ends of the trench. The pipe is subsequently removed when the concrete has partially cured, leaving a Semi-Circular joint between panels. The surface of the joints from the inside is treated with epoxy resin

1. First one end, then opposite end of panel is excavated in full depth. During this time, the excavation is kept filled with bentonite suspension.

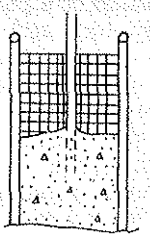


2. Centre of panel is excavated.

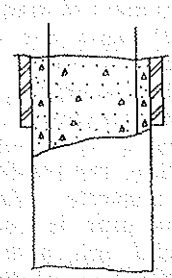


backed strips for water-proofing. Otherwise, the joints can be gunited or grouted behind the wall.

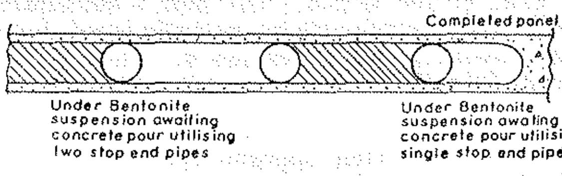
5.2.4.1.2. The various sequences in the construction of such R.C.C. cut-offs are shown below by sketches :



3. Concreting of penals. Steel stop end pipes and reinforcement case positioned and concrete placed through tremie pipe. The bentonite is displaced by the concrete.



4. Section through completed panel showing guide walls and steel.



5. Series panels on plan illustrating uses of steel stop-end pipes as required.

Various Sequences in the construction on such R.C.C. Cut offs

5.2.5. *Pier*

Piers are provided between each bay. The gates operate through the grooves provided in the piers. Usually there are two grooves, the upstream one called the stoplog groove and downstream one called the main gate or service gate groove. Sometimes there may be a third groove also for emergency purposes. In the case of head regulators, there will be additional inclined grooves on the upstream end through which trash racks are installed wherever they are required. The road bridge and/or railway bridge are laid from one bank to the other over the piers and the bearings are installed on top of the piers. The bridge for the gate hoisting arrangement is also taken over the piers. Wherever necessary dummy piers are provided in between the main piers to load down the raft against uplift pressure. Sometimes to reduce the size of the gates, dummy piers are introduced between the main piers.

5.2.5.1. The piers are constructed either monolithic with the floor (raft) or made independent with their own foundation footings. In the latter case, seals will be provided all around to prevent leakage between the floor and the pier (See Figure No. 5 6).

5.2.5.2. The thickness of pier varies from 1.5 m to 2.5 m generally. The thickness should be adequate to resist the moments created (discussed in more details in the chapter on Structural Designs) and also accommodate the embedded parts for the Main gate and stoplog grooves.

5.2.5.3. Wherever reinforced concrete rafts are provided, the pier extends usually from the upstream end to the dowstream end of the floor to reduce the span length of the raft. The upstream cut-water usually has a semicircular shape and downstream ease-water usually has an equilateral shape with circular curves. The upstream and downstream vertical ends are sometimes given a better of 1 in 18-20 depending on the architecture. Sometimes, a few project authorities prefer to have square ended cut-waters and ease-waters While these may be done above the high flood levels if aesthetic value is not disturbed, they should not be done below the flood levels so that hydraulic efficiency is not impaired.

5.2.5.4. The piers have to be high enough to hold the gates clear off the maximum flood while making ample allowance for passing the floating debris under the gate. The height can be reduced by using double or triple gates. The height of the piers in the zones away from gate bridge, road bridge railway bridge and instrumentation terminal panel location zones, can be reduced if submergence of the piers is permitted. However, the height of the piers should also be viewed from any additional dead weight required while computing the overall stability of the structure. The height of the pier in the zone where gate hoisting arrangements are located is usually higher than the other zones and is constructed with the same material as the rest of the pier, *i.e.*, concrete or masonry. But in Earthquake regions, constructing higher portion of the pier with concrete or masonry is to be avoided so that extra earthquake horizontal moments are avoid. Instead, steel trestles of required height can be erected so that the moments are reduced due to light weight. structure. However, this again is a question of comparison of cost between the two alternatives. Sometimes for repair of the gate wheels, etc., a gallery is constructed at higher levels in the pier with access from the road bridge over the piers.

5.2.6. *Divide wall*

The divide wall is also like a pier and is provided between undersluice and river-sluice or undersluice and spillway or river-sluice and spillway as the case may be. The following are its main functions :

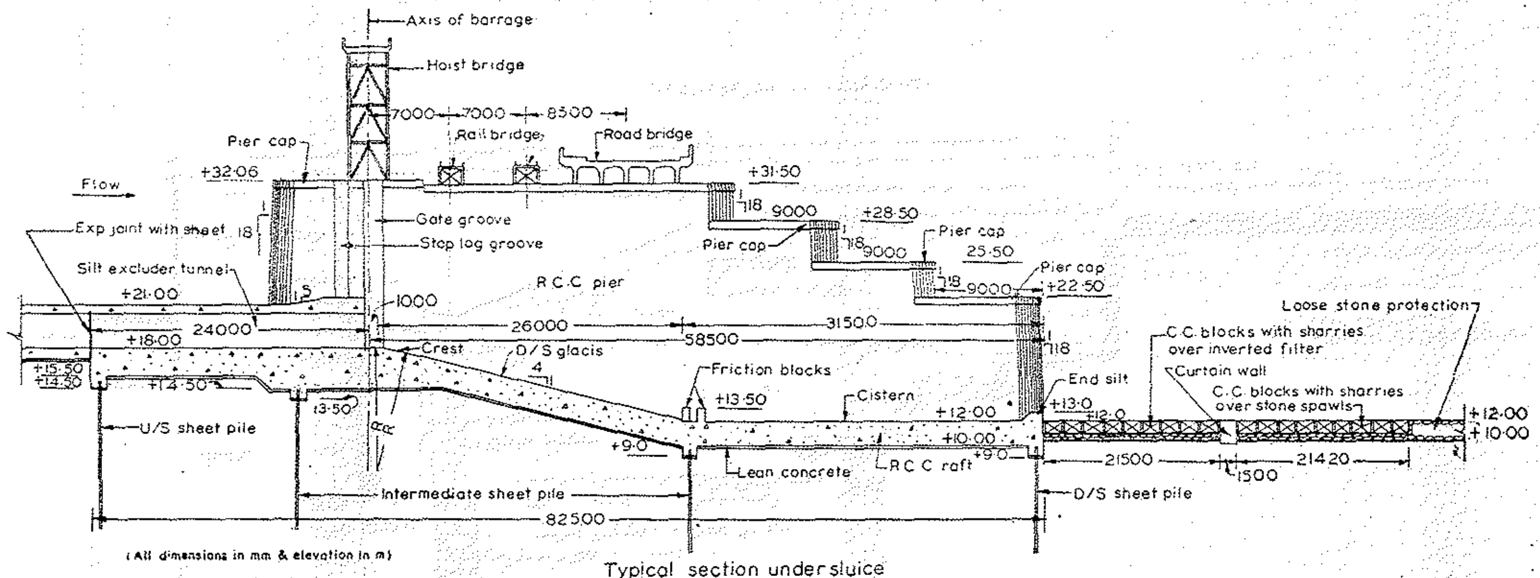


FIGURE 5.4 : Typical Section of Undersluice with Silt Excluder bay.

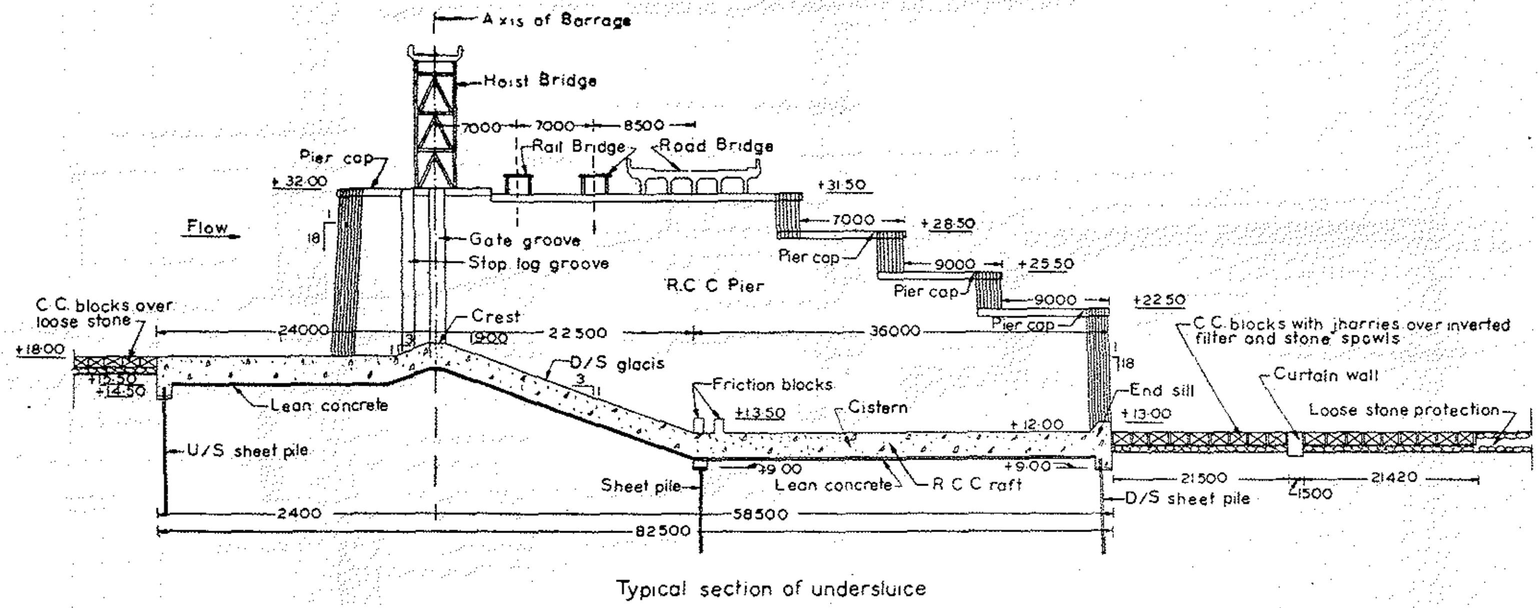


FIGURE 5.5 : Typical Section of Undersluice without Silt Excluder bay.

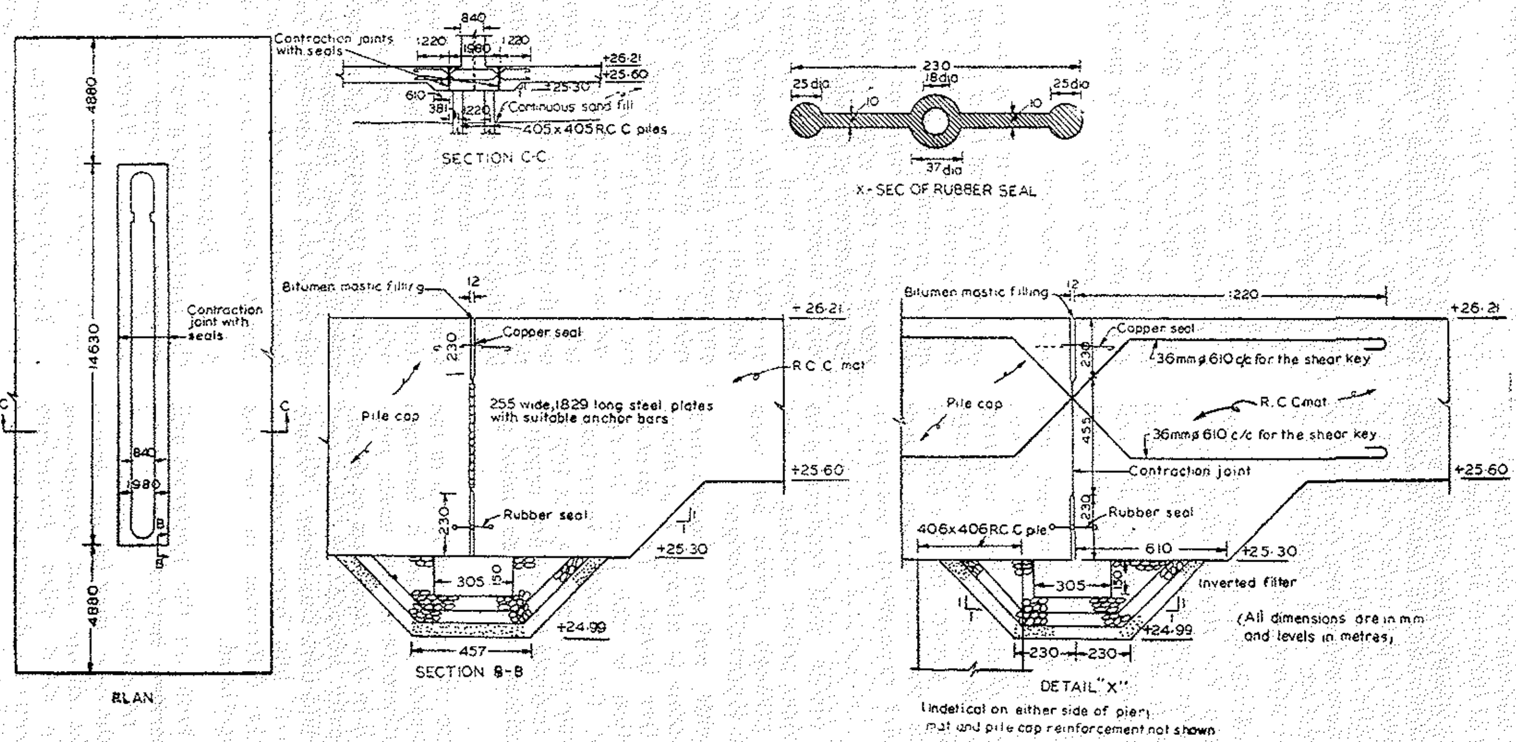


FIGURE 5.6 : Typical details of Seals Around pier Footings.

(*i*) Its separates the turbulent flood waters from the pocket in front of the canal head. It also prevents heavy disturbance which would otherwise result on account of the water in the two portions being at different levels and

(*ii*) It helps in checking parallel flow which would be caused by the formation of deep channels leading from the river to the pocket in front of the sluices. If parallel currents still form, the divide wall keeps them away from the floor of the diversion structure and upstream and downstream flexible protections.

5.2.6.1. The length of the divide wall on the upstream has to be such as to keep the heavy action on the nose of the wall away from the upstream protection of the sluices and also to provide deep still water pond in front of the canal had regulator. The divide wall also plays an important part in the control of silt entry into the canal by enclosing a pocket of nearly still water and by separating it from turbulence and vagaries of the main river. Similarly on the downstream side, it should be sufficiently long to guard against action set up by the undersluice discharge damaging the spillway flexible apron on the side of the divide wall. Sometimes the downstream divide wall can be omitted if model experiments indicate to that effect.

5.2.6.2. The length of the divide wall required to be provided initially in the designs before testing it for its adequacy or otherwise through models has been discussed in the Chapter on Hydraulic Designs.

5.2.6.3. The top of the divide wall on the upstream side near the diversion structure is always kept above the pond level with some free board. This gives an indication of the obstruction in the pond for any crafts coming into the pond for sounding, inspection, recreation etc. Beyond some safe distance, the top of divide wall can be lowered (subject to stability requirements). However, any decision should be based on the model test results.

5.2.6.4. Due to difference in the bed levels in the spillway/Riversluice/undersluice portions, it becomes necessary to negotiate the difference in the levels over the length of the divide wall in the upstream or downstream side. The slope usually adopted is 1 in 5 and the flexible protections are laid over this slope. Where the downstream divide wall has been omitted on relevant considerations, the difference in the bed levels has to be suitably negotiated.

5.2.6.5. The divide walls both on the upstream and downstream sides, are separated beyond the raft by joints and have their own foundations. Concrete cut offs or sheet piles, as the case may be, are driven along both the faces of the divide wall foundations and joined up with the main sheet piles. Sometimes some portion of the upstream or downstream ends of the divide wall rest over well foundation as in the case of Wazirabad Barrage in Delhi. (See Figure 5·7). The shank and nose of divide walls should be properly protected as per the criteria discussed in the chapter on Hydraulic Designs.

5.2.7. *Double Pier*

In a very long structure, double piers are introduced at specified intervals. These are nothing but two single piers with a joint in between them and also provided with seals both vertically and horizontally (See Figure 5·8) for typical details. Double piers are introduced for a number of reasons such as change in the foundation characteristics, construction programme and length of the floor raft which can be conveniently and economically cast, etc. The spacing generally varies from 8 to 12 bays though smaller spacings are also there due to the above mentioned reasons. In a way, divide walls are only extension of the double piers between the sluice bays and spillway bays.

5.2.8. *Abutment*

The abutments form the end structures of the diversion structure and their layout depends on the project features and topography of site. The length of the abutment is generally kept the same as the length of the floor. The top of the abutment will be fixed with adequate free board over the upstream and downstream water levels. From upstream to downstream the top of the abutment can be made slopping or stepped. The thickness of the abutment must be adequate to accommodate the grooves and bridge bearings also. Due to the different intensities of loading and to avoid damages due to any possible differential settlement, generally the abutments are divided into different blocks such as upstream, gate bridge, road bridge, downstream blocks, etc. Each block is separated from the others by joints and provided with seals both vertically and horizontally. The toe slab of toe foundation of the abutment usually forms part of the raft of the end bay adjacent to the abutment and separated from the raft by a longitudinal joint provided with seals. The foundation of the abutments should be carried atleast to the same levels as those of the floor of the Main diversion structure and preferably a little lower. The foundation of the abutments should provide a boxing of the diversion structure. Cross-cut-off or sheet piles are continued from upstream to the downstream end of the floor and well connected to the main upstream and downstream cut-offs or sheet pile lines. Typical details of abutment are given in Figure 5.9.

5.2.8.1. Sometimes, in the case of small structures, the abutments along with the floor of the bays can be designed as a trough section. Typical layout with section is shown is Figure 5.10.

5.2.9. *Flank Wall*

In continuation of the abutments of the diversion structure, flank walls are provided both on the upstream and downstream sides on both the banks. The flank walls ensure smooth entry and exit of water

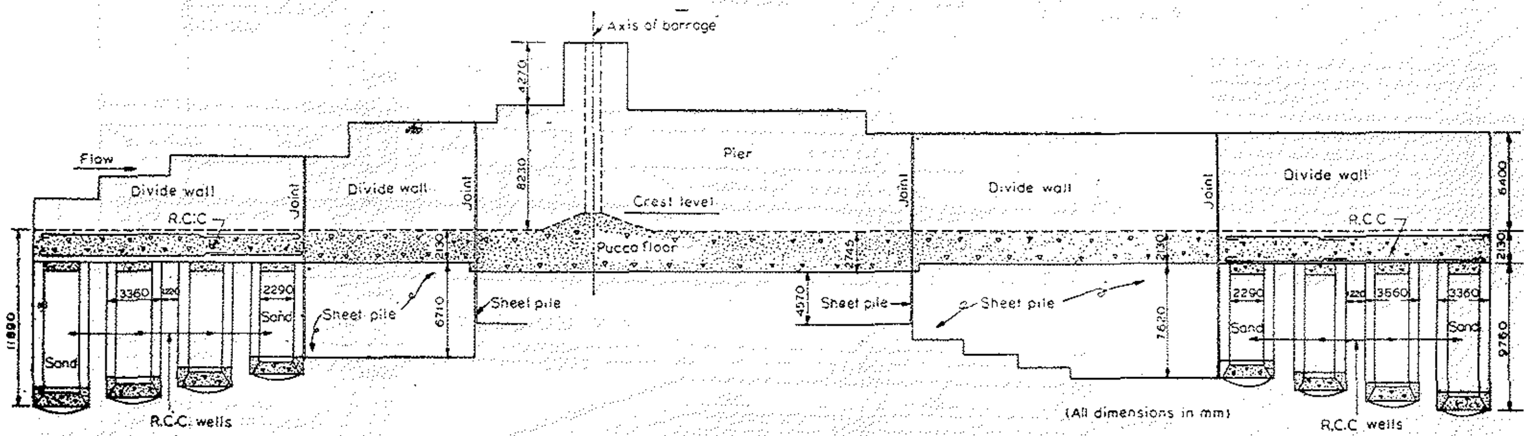


FIGURE 5.7 : Details of divide walls of Wazirabad Barrage.

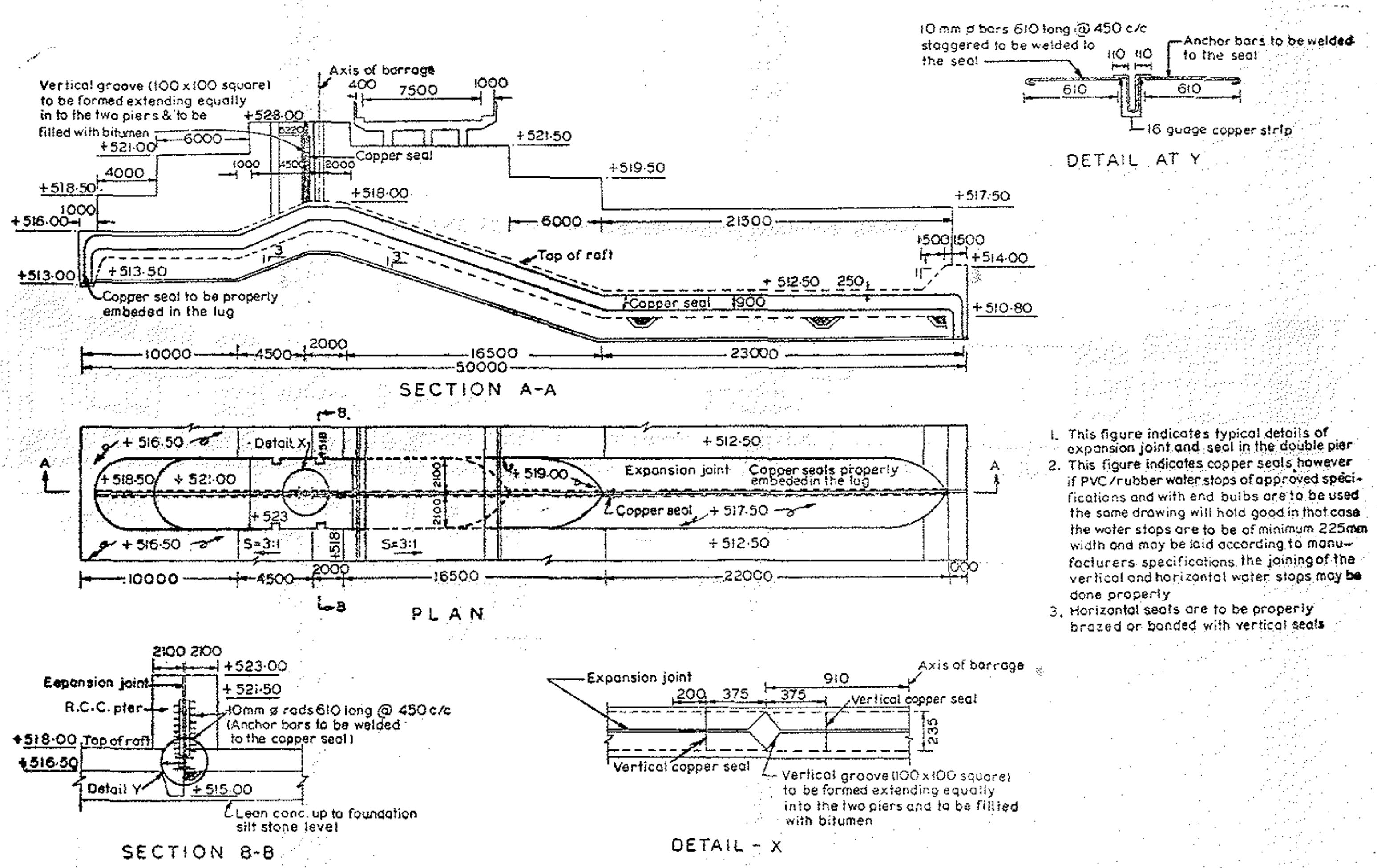


FIGURE 5.8 : Typical details of Double pier.

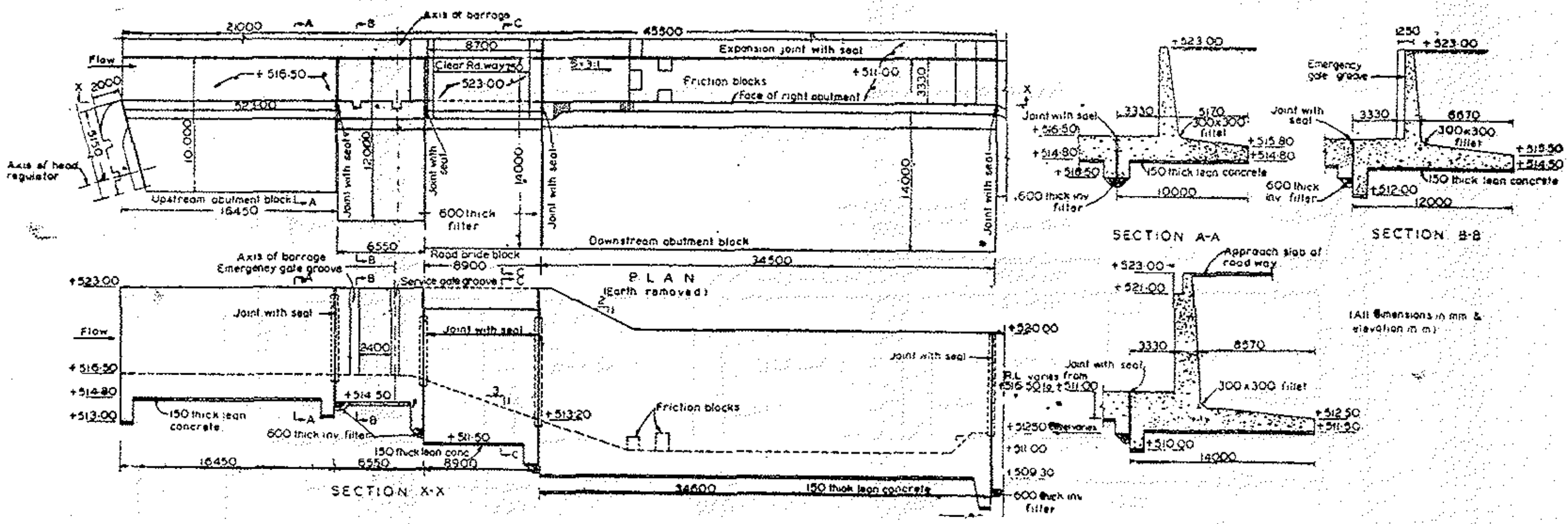


FIGURE 5.9 : Typical details of Abutment.

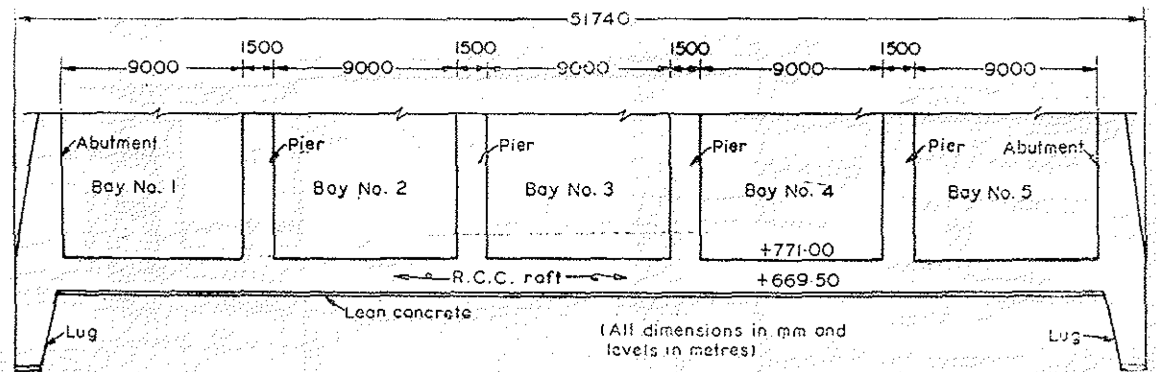


FIGURE 5.10 : Typical details of Abutment as Trough Section.

and away from the diversion structure. The flank walls are generally laid out to provide a transition of 1 in 2 to 1 in 3 from the guide bunds to the barrage proper. The total transition length is made up of two types of construction, one with solid concrete/masonry wall, usually called the flank wall and the other with concrete/masonry and C.C. blocks, usually called the flared out wall. The water face of the flank wall (solid concrete wall) is generally changed from vertical at the abutment end to a slope of 0·5 (horizontal) : 1 (vertical) and the steam is constructed of concrete/masonry throughout its height. The water face of the flared out wall is generally changed from the slope of the end section of flank wall (generally 0·5 : 1) to the slope of the guide bunds which is generally 2 : 1 to 3 : 1. The stem of the flared out wall is constructed of concrete/masonry for certain height, overlaid by interlocking cement concrete blocks one above the other. The number of interlocking cement concrete blocks varies as the slope of the flared out wall varies. The flared out wall is also constructed in blocks with joints in between. At the back of the flank wall, arrangements are usually made for pressure release. Typical details of flank walls are given in Figure 5.11.

5.2.10. *Return Wall*

Return walls are generally provided at right angles to the abutment either at its ends or at the flank wall portion. (See Figure 5·12). Wherever return walls are constructed as part of the flank wall it is desirable to separate the stem of the return wall from the steam of the flank wall by a joint. The base slab for the flank wall and return wall would be the same

5.2.10.1. Where the return wall are provided at the ends of the abutments, they are extended atleast upto the end of the heel portion of the base slab of upstream or downstream abutment blocks as the case may be. In the case of the head regulator, the return walls are to be keyed properly into the embankment of the canal for a length of at least 2 metres beyond the top edge. Where the return wall is part of the flank wall, the same is extended upto the end of the heel portion of the base slab.

5.2.11. *Guide Bunds*

It is necessary to narrow down and restrict wide alluvial river course to flow axially through the diversion structure. Guide bunds are provided for this purpose of guiding the river flow past of diversion structure without causing damage to it and its approaches. The layout and design of guide bunds have been discussed in detail in Chapter 2, Volume II River Training Works.

5.2.12. *Afflux Bunds*

Afflux bunds are components of the diversion strucpures wherever necessary to protect important fow lying troperties adjacent to the structures from submergence due to affluxed high floods. For more details reference may be made to Chapter 2, Volume II.

5.2.13. *Approach Embankments*

Diversion structures are constructed with a restricted waterway for economy in wide alluvial rivers and better flow conditions. The unbridged width of the river is blocked by means of Approach embankments. For more details, reference may be made to Chapter 2, Volume II.

5.2.14. *Silt Excluding Devices*

It is never possible and usually never necessary to eliminate the sediment completely from a channel. However, it becomes necessary sometimes to remove portion of the silt in the water depending on the specific purpose for which water is required through the canals. The various devices used for controlling sediment are classified as preventive and curative. As the manual is on Barrage and weirs, the topic is confined only to preventive measures and sediment or silt excluders come under this category. It is a device placed in front of the undersluice and head regulator to exclude the lower layers of water which carry the comparatively coarser particles and prevent them from being diverted into the canal. The silt excluder is generally in the form of tunnels. In bouldery reaches, where the slope of the river is rather high, the river carries down big sized boulders and if a high pond is contemplated, it may be satisfactory to provide silt excluder tunnels. In those cases, silt deflecting arrangement with open top is preferable. The details of silt exclusion has separately been dealt with in Chapter 1 of Volume II on Silt Exclusion wherein the different types and layouts have also been discussed. Hence these are not elaborated here.

5.2.15. *Fish Pass*

In some of the diversion structures, the need for providing suitably designed fish pass arises. The fish pass enables the migratory fish to move across the diversion structure from downstream to upstream and vice versa. The fish pass may be in the form of a ladder or lock. Fish passes are generally located near the divide walls due to the availability of water throughout the year in the river downstream of the undersluices. It is provided between the spillway and sluice bays. They can also be located at one or both of the abutments. The various types and layouts have been discussed in Chapter 3, Volume II on Fish pass.

5.2.16. *Navigation Lock*

With inland navigation gaining importance, wherever feasible, navigation facilities are combined with the diversion structures. This includes provision of a navigation channel with navigation locks suitably incorporated to allow passage of crafts to move from upstream to downstream and vice versa and through

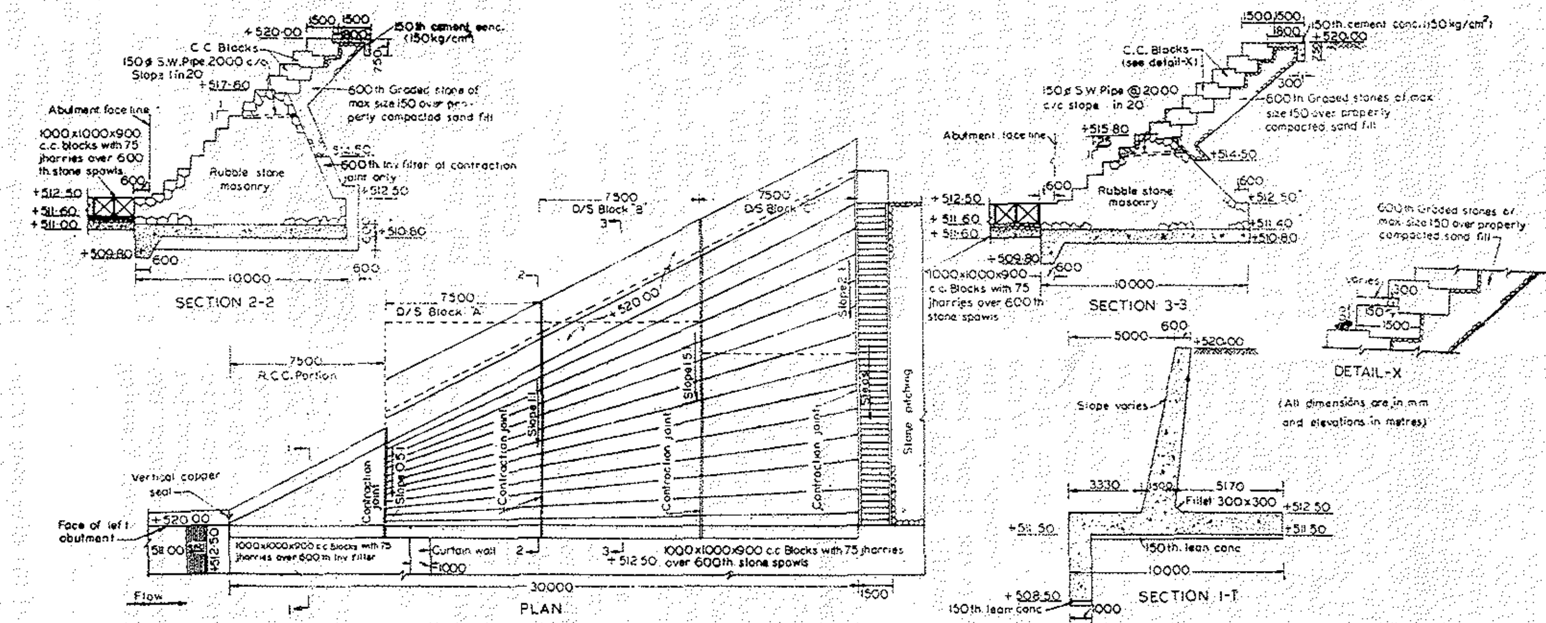


FIGURE 5.11 : Typical details of Flank Walls.

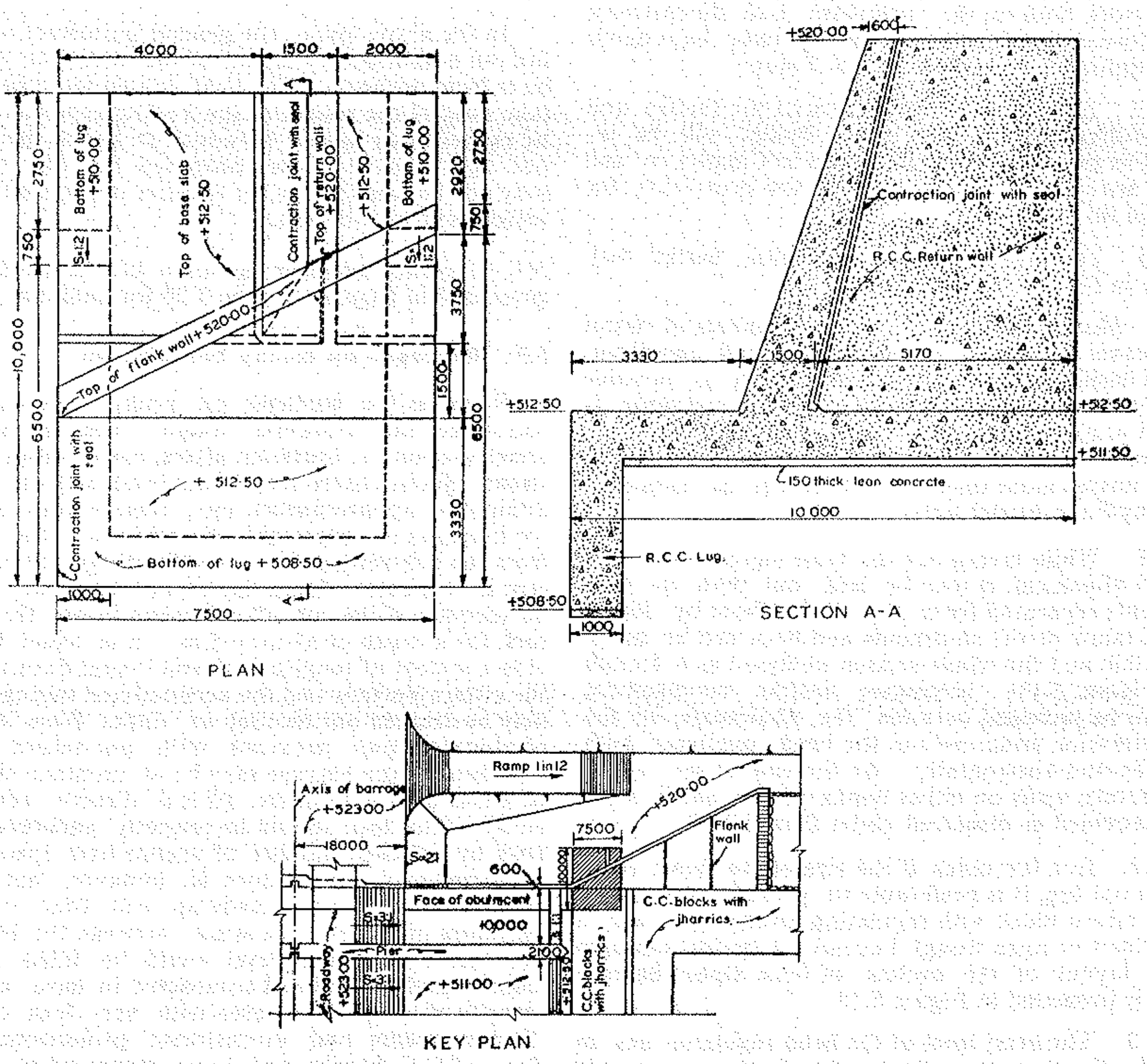


FIGURE 5.12 : Typical details of Return Wall.

a feeder canal also if desired. Necessary berthing facilities and repair facilities are also to be provided. The navigation channel is generally taken out on the right or left bank, as the case may be, away from the diversion structure and sometimes it crosses the main canal from the diversion structure also. Layout of the navigation arrangement has to be done as per requirement of size of craft, etc. This is discussed in more detail in Chapter 4 of Volume II on Navigation Facilities.

5.2.17. *Bed Protection*

To protect the diversion structure from damages likely to be caused by the scouring of the river bed both on the upstream and downsteam due to the flow through the structure, bed protection is to be provided. This is usually in the form of cement concrete blocks over stone spawls or inverted filter and loose stone of adequate thickness. On the upstream side, immediately in front of the floor, cement concrete blocks over stone spawls are laid. Further upstream loose stone protection is provided. The length and thickness of each is determined as discussed in Chapter 6 on Hydraulic Designs. Usually the size of the cement concrete block is 1500 ×900 mm. The thickness of stone spawls is usually 600 m and that of loose stone protection in wire crates 1500 mm on the downstream side, the total length of c.c. block protection estimated as discussed in Chapter 6 is provided, half over inverted filter and half over stone spawls and separated by a curtain wall. On the downstream side, for the release of any pressure, 75 mm thick jharries are provided between the c.c. blocks and these are filled with bajri. Beyond c.c. block protection, loose stone protection in wire crates are laid for the estimated le ngth.

5.2.17.1. Any difference in the levels of these protection work both on the upstream and downstream sides between the sluices bays and spillway bays needs to be negotiated, usually over 1 in 5 slope.

5.2.17.2. The bed protections around the shanks and noses of divide walls and guide bunds will be increased as per the criteria discussed in Chapter 6 and will be provided in continuation of the protection for the main floor.

5.2.17.3. The layout of these protection works may be seen in Figures 5.14 to 5.20.

5.2.18. *Head Regulator* : The requirements of water in the canal are drawn through the head regulator. As it is desirable to exclude silt as much as possible from the head regulator, axis of the head regulator is laid out at right angles to the axis of the diversion structure. However, sometimes it may have to be kept at angles more than 90°. This will be indicated by the hydralic model tests.

5.2.18.1. While laying out the head regulator at the selected alignment, it may be laid out with its own abutments separated from the main floor by longitudinal joints or the abutments and floor can be made monolithic and the whole section analysed as a trough (See Figure 5.13). Necessary sealing arrangement needs to be provided between the abutments of the main diversion structure and the head regulator both vertically and horizontally. At the end of the abutment, return walls on either banks of the canal have to be provided as discussed under Para 5.2.10.

5.2.18.2. In a few cases, if the size of the head regulator is not big, it is preferable to provide the outlet with a return block with regulating arrangement and a box culvert or open trough beyond. A typical example is the layout of the outlets of Iram Siphai Barrage which is presented in Figure 5.18.

5.2.18.3. The crest level of the head regulator has to be high enough so that silt is not drawn excessively through the structure. Economy in cost of the gates is acheived by the provision of breast wall above the top of the gate. The cut-off of the regulator on the upstream side has to be taken down in conjuction with the cut-off of the upstream block of the abutment of the diversion structure such that there is no short-circuiting of the sub-soil flow.

5.2.18.4. A detailed discussion on the head regulator is presented in Chapter 5, Volume II.

5.2.19. *Regulating Arrangement*

In the diversion structure, the river flow is regulated by means of gates with their hoisting arrangement. Similarly the canal flow through the head regulator is also regulated by gates. The success or failure of the diversion structure will depend on the satisfactory installation of these. These are discussed in Chapter 6, Volume II on Gates, Stoploge and Trash Rack.

5.3. General

In the above paras, the general guidelines for working out a satisfactory layout of the diversion structure have been outlined. The final layout adopted should take note of the results of the 3—Dimensional hydraulic model tests also. However, it has to be pointed out again that a layout proposed for one structure may not be satisfactory for another one since the parameters will be different.

5.4. Layouts adopted in some of the projects are presented in Figures 5.14 to 5.20 for guidance.

5.5. Barrages on Rocky Foundation

So far as the barrages on rocky foundation are concerned, the hydraulic designs from surface flow consideration i.e. upstream glacis, crest width, downstream glacis, cistern length and level, end sill, energy dissipation appurtenances, etc., remain same as those for barrages on permeable foundation. The designs from the sub-surface flow considerations will be different in this case. Since there won't be scour, it may be adequate if the cut offs are taken inside the sound rock for a depth of about 0·5 m. It is usual to provide a system of longitudinal and lateral drains below the cistern portion and the accumulated seepage water may be drained out through an outlet from the face of abutment/pier provided with non-return valves. The floor of the barrage may be of nominal thickness of about 0·5—0·75 metre (if it is directly resting on rock). The floor should be properly anchored to the rock by sufficient number of anchor bars spaced in a staggered way. If the floor is, however, not resting over the rock, but on made-up soil (due to higher elevation of cistern), the space between the bottom of the floor and the rock level could be filled up with coarse sand, wetted and compacted to have a relative density of about 0.7 or even with very lean concrere. The upstream and downstream protections in the form of C.C. blocks and loose stone apron may be dispensed with if the bed is not erodible. However, where the rock is jointed and weathered, some scour is likely to occur. Hence, in those cases, it is always preferable to provide c c. block protection for a nominal length.

5.6. Barrages on Clayey Strata

5.6.1. It is not always the case that a barrage is resting on permeable foundation. Often times, it is seen that the barrage may have to be founded on clayey strata. During the excavation, if clay patches of small thicknesses are encountered, usually these patches are completely removed and replaced by sandy material and compacted to the desired relative density (usually 0.7). However, if the clayey material is extensively present to a large depth as revealed by boreholes, then the barrage structure has to be very carefully designed and constructed.

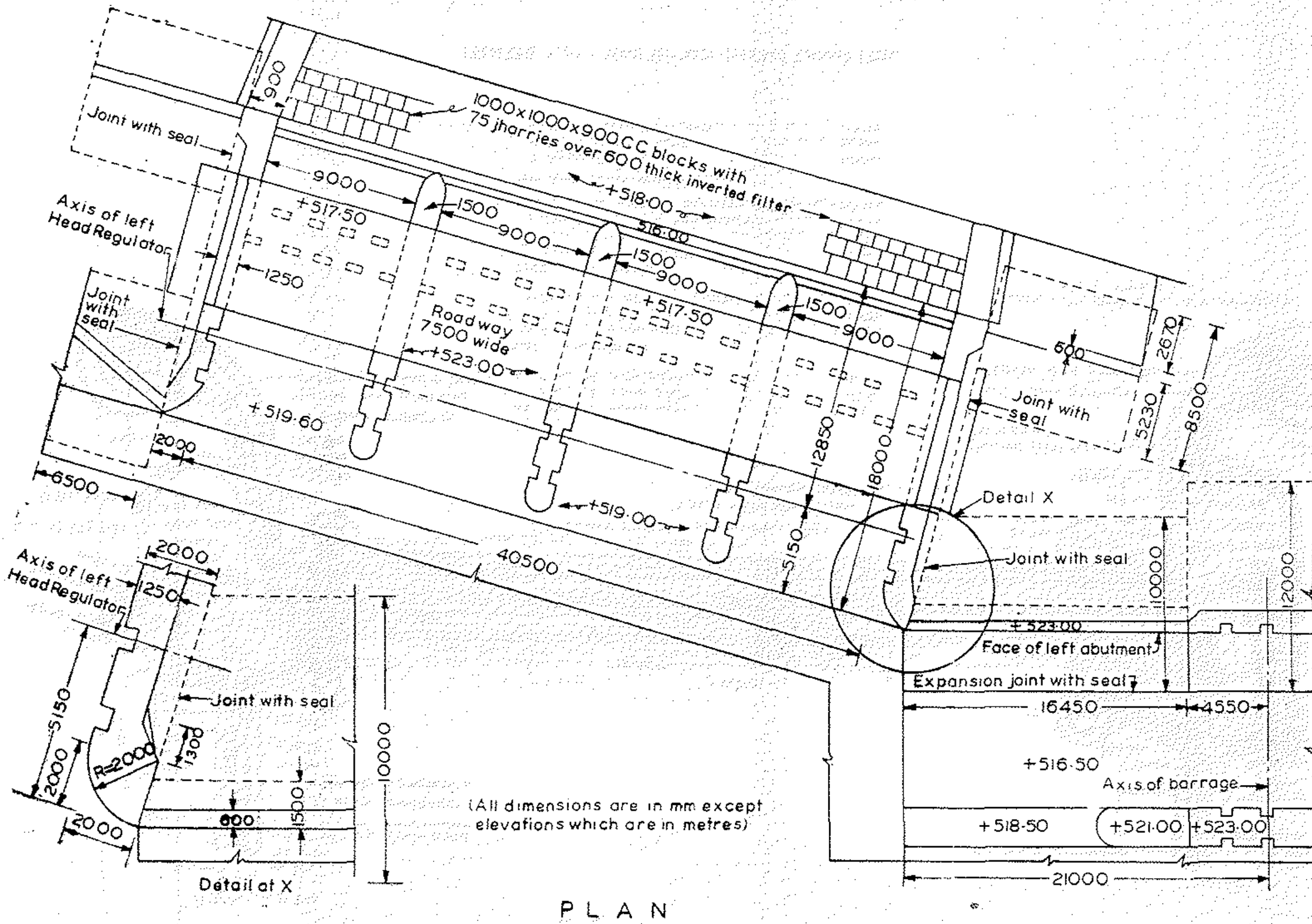


FIGURE 5.13 : Typical Layout Head Regulator.

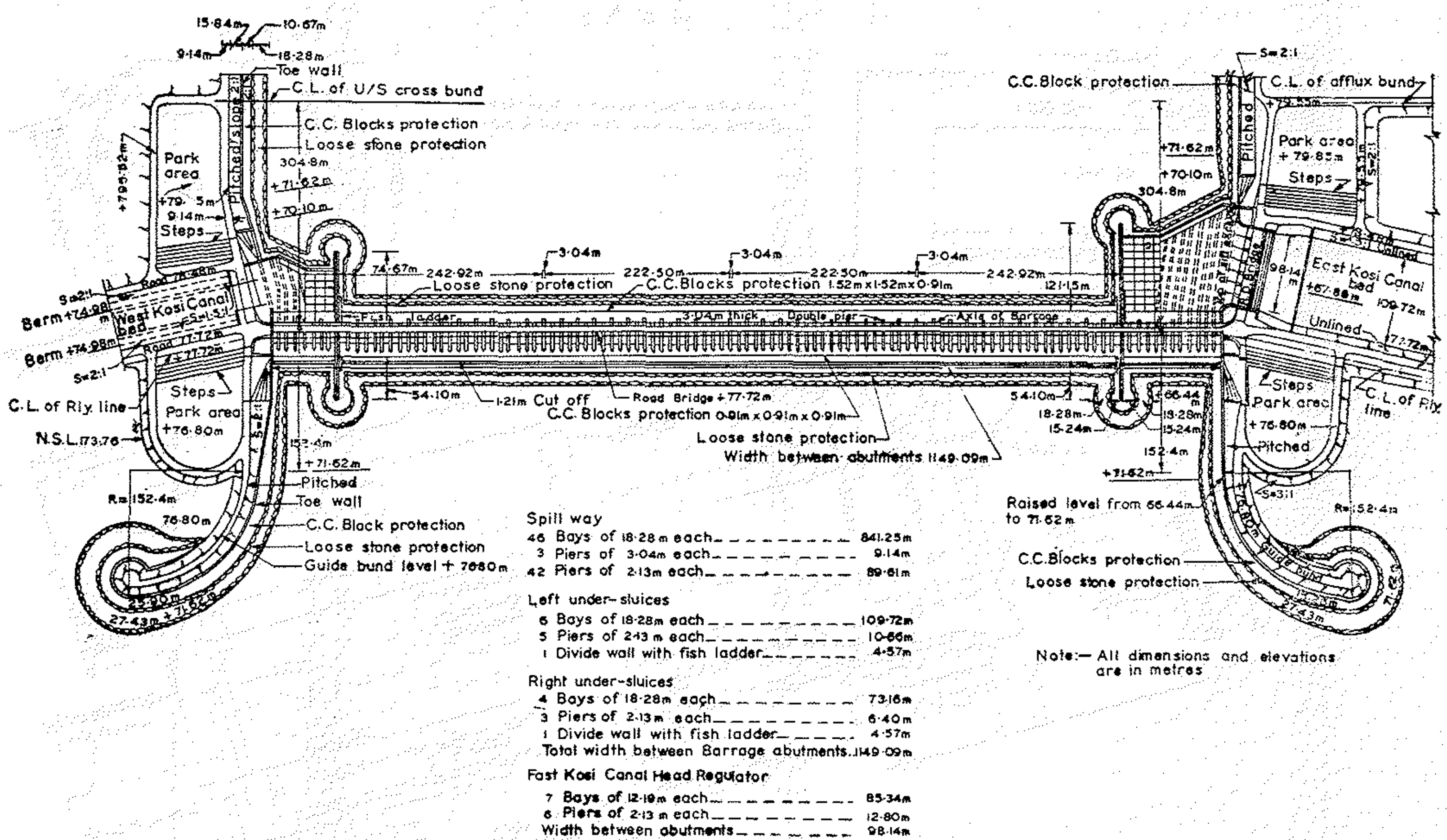
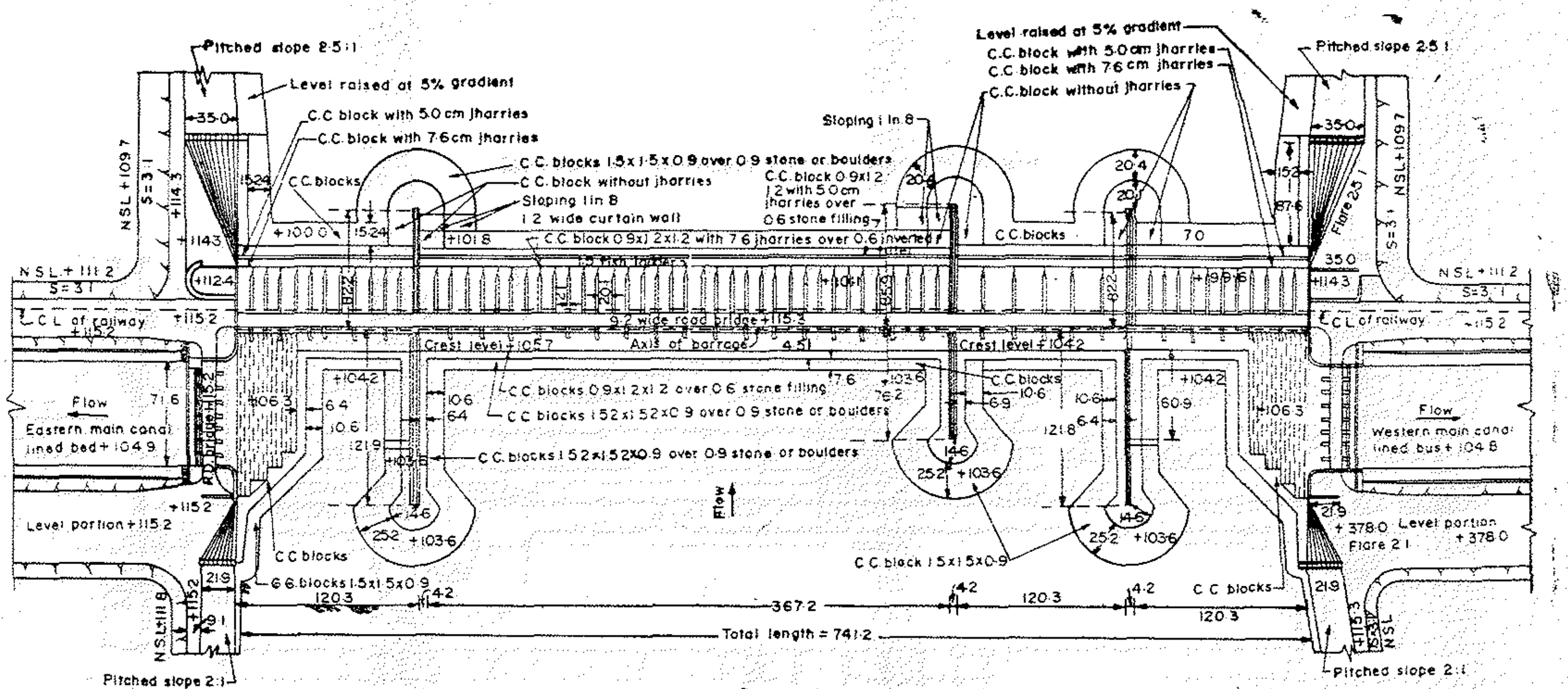


FIGURE 5.14 : Kosi Barrage detailed Layout Plan.



Barrage features

				B F	4962
1 Left under sluices			5 River sluice		
6 spans of 18.2 each	109.7		6 spans of 18.2 each	109.7	
5 piers of 2.13 each	10.6		5 piers of 2.13 each	10.6	
	120.3	120.3		120.3	120.3
2 Left divide wall			6 Right divide wall		
Divide wall	2.13		Divide wall	2.13	
Divide wall pier	2.13		Divide wall pier	2.13	
	4.26	4.26		4.26	4.26
3 Spillway			7 Right under sluices		
18 spans of 18.2 each	329.2		6 spans of 18.2 each	109.7	
15 piers of 2.13 each	32.0		5 piers of 2.13 each	10.6	
2 double pier of 3.0 each	6.0			120.3	120.3
	367.2	367.2	Width between abutments		741.2
4 Divide wall between spillway and river sluice			8 Eastern and Western head regulators		
Divide wall	2.13		8 spans of 7.6 each	60.9	
Divide wall pier	2.13		7 piers of 1.5 each	10.6	
	4.26	4.26		71.5	
C.O		4962	Width between abutment of the head regulator		71.5

Note:
All dimensions and elevations are in metres

FIGURE 5.15 : Gandak Barrage detailed layout plan.

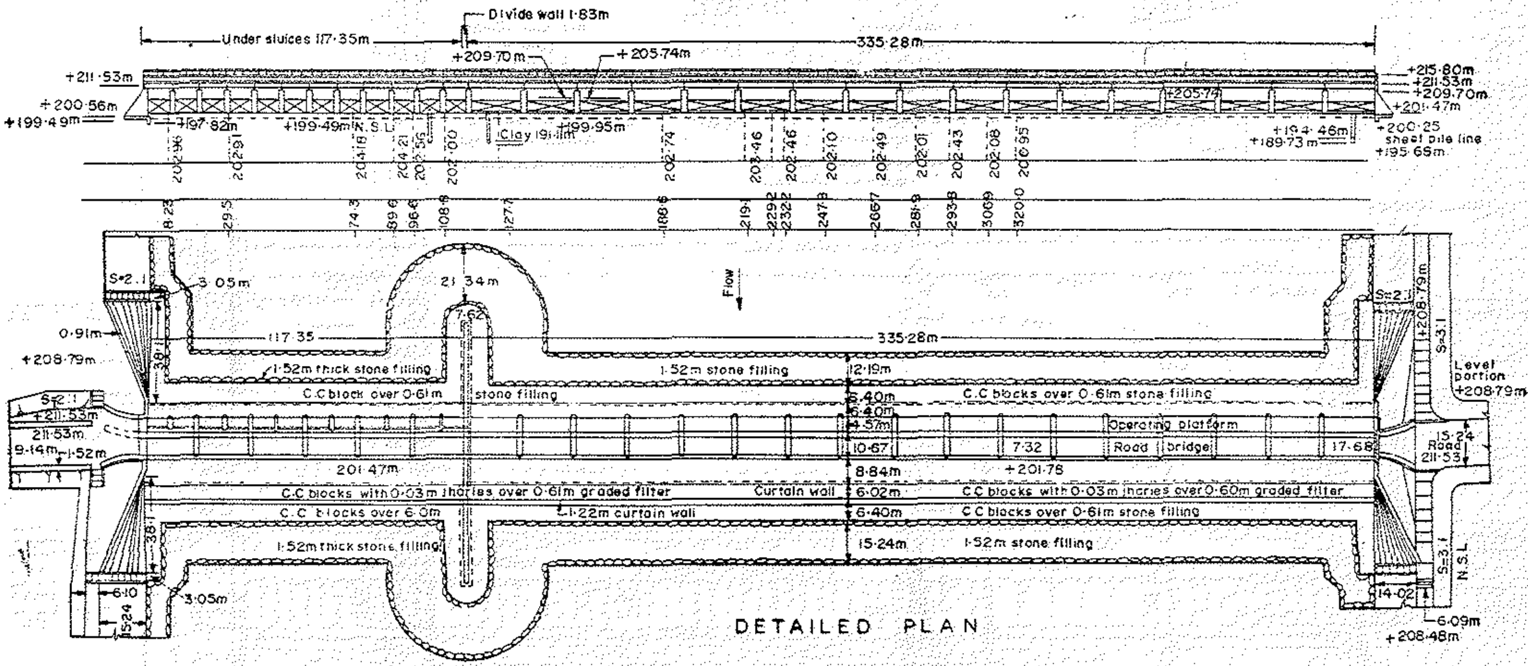


FIGURE 5.16 : Wazirabad Barrage detailed Layout Plan.

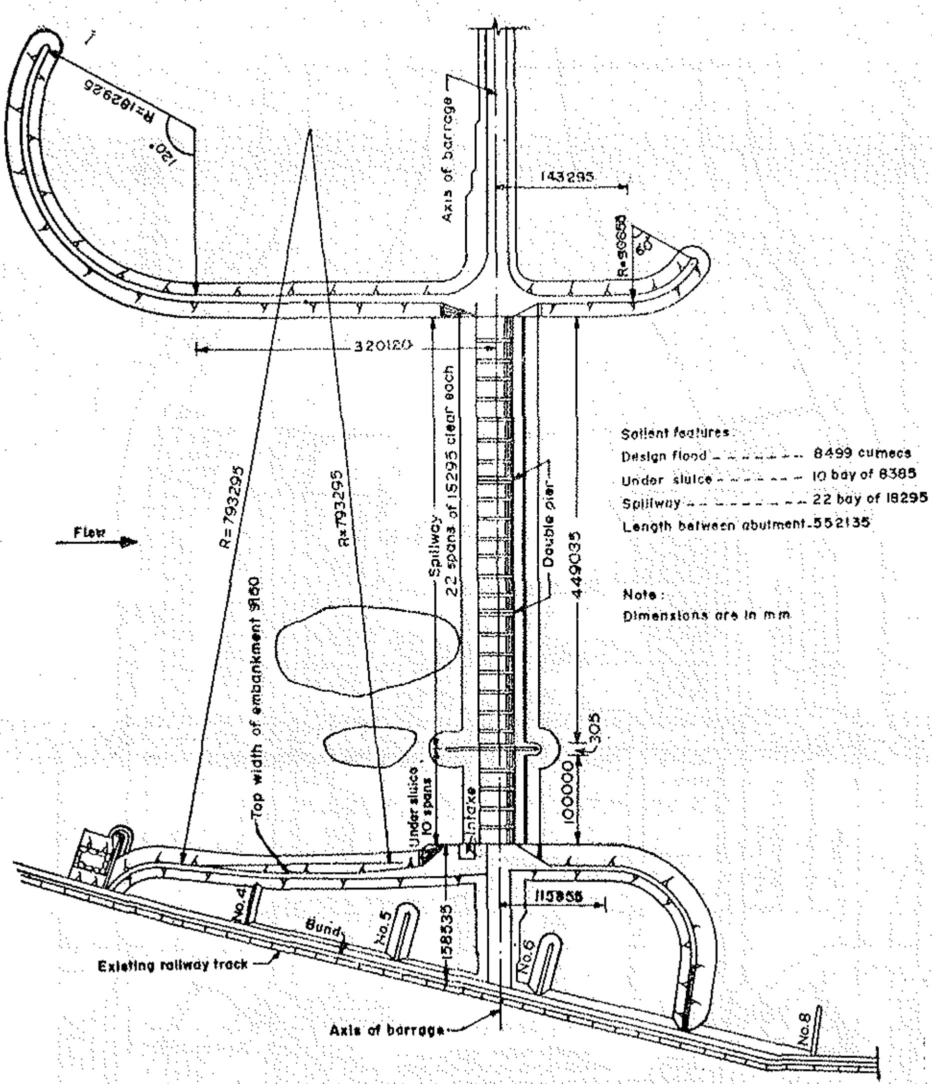


FIGURE 5.17 : Detailed Layout Plan of Yamuna Barrage.

5.6.2. The waterway for a barrage on clayey reach may have to be provided as governed by the site conditions. A study of the cross-section at the barrage axis will give the necessary guidance in this respect.

5.6.3. In a case of clayey foundations, it is difficult to evaluate the anticipated scour depths. The resistance to scour offered by the cohesion of clay would initially reduce the scour. However, over a long period, when this resistence is overcome, total scour could be anticipated. Hence, generally the scour depths are estimated based on the soil properties overlying the clayey strata.

5.6.4. The factors that require careful consideration in respect of structure on clayey strata are (*i*) adequacy of the structure against sliding (*ii*) permissible bearing pressures and (*iii*) probable settlements due to load conditions imposed on the strata by the structure. In order to determine all these and have a review, data has to be very carefully collected in the field and analysed in the laboratory.

5.6.5. *Data to be Collected*

Soil samples should be carefully taken in the clayey strata from a number of bore holes, mostly along the axis and on the upstream and downstream cut off/sheet pile lines and at different depths also. The laboratory analysis should indicate their engineering properties such as liquid limit, plastic limit, grain-size gradation, in-situ density, field moisture content, void ratio, pre-consolidation pressure, compression index, e-log p relationship, etc.

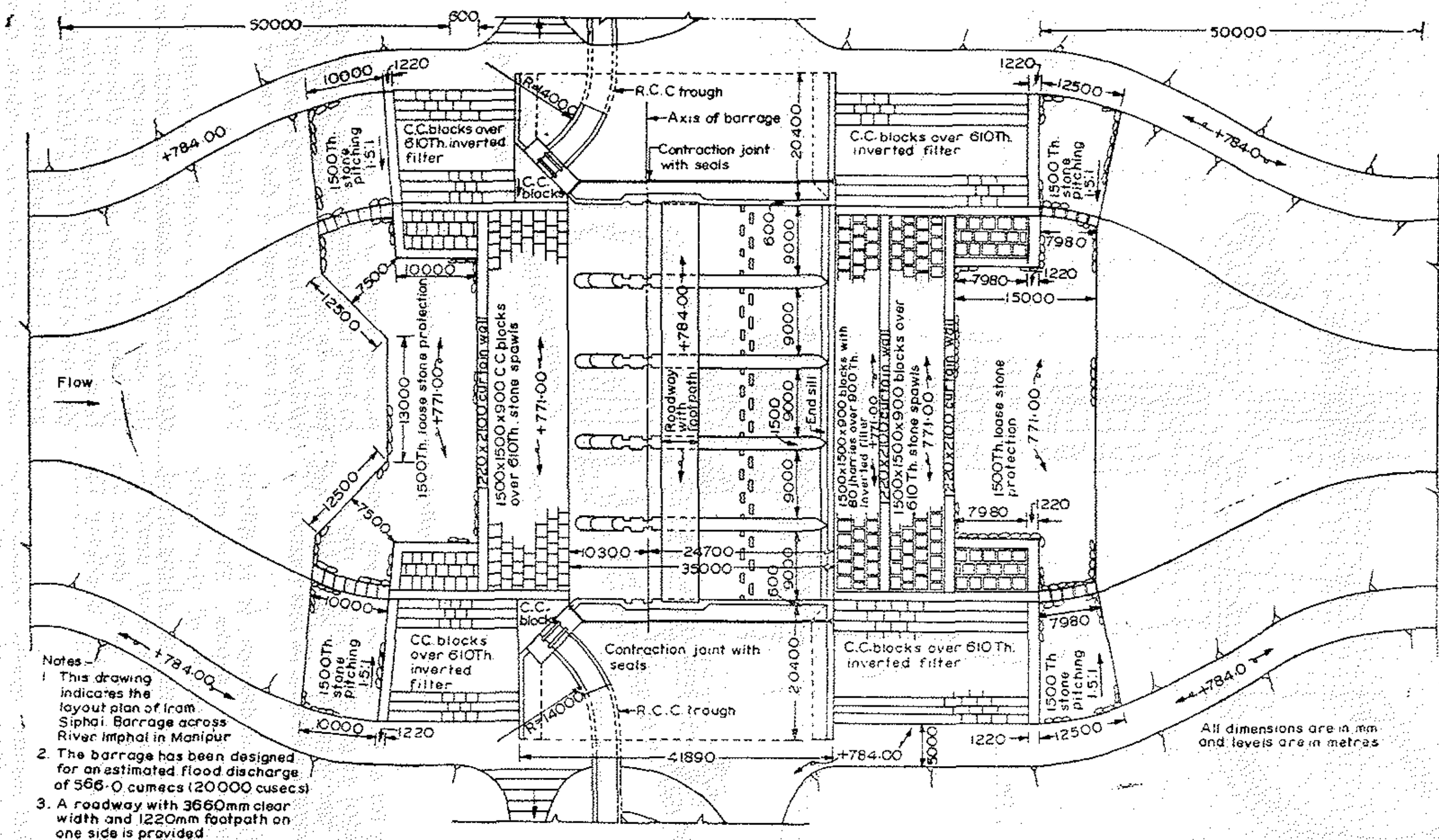


FIGURE 5.18 : Detailed Layout plan of Iram Siphai Barrage.

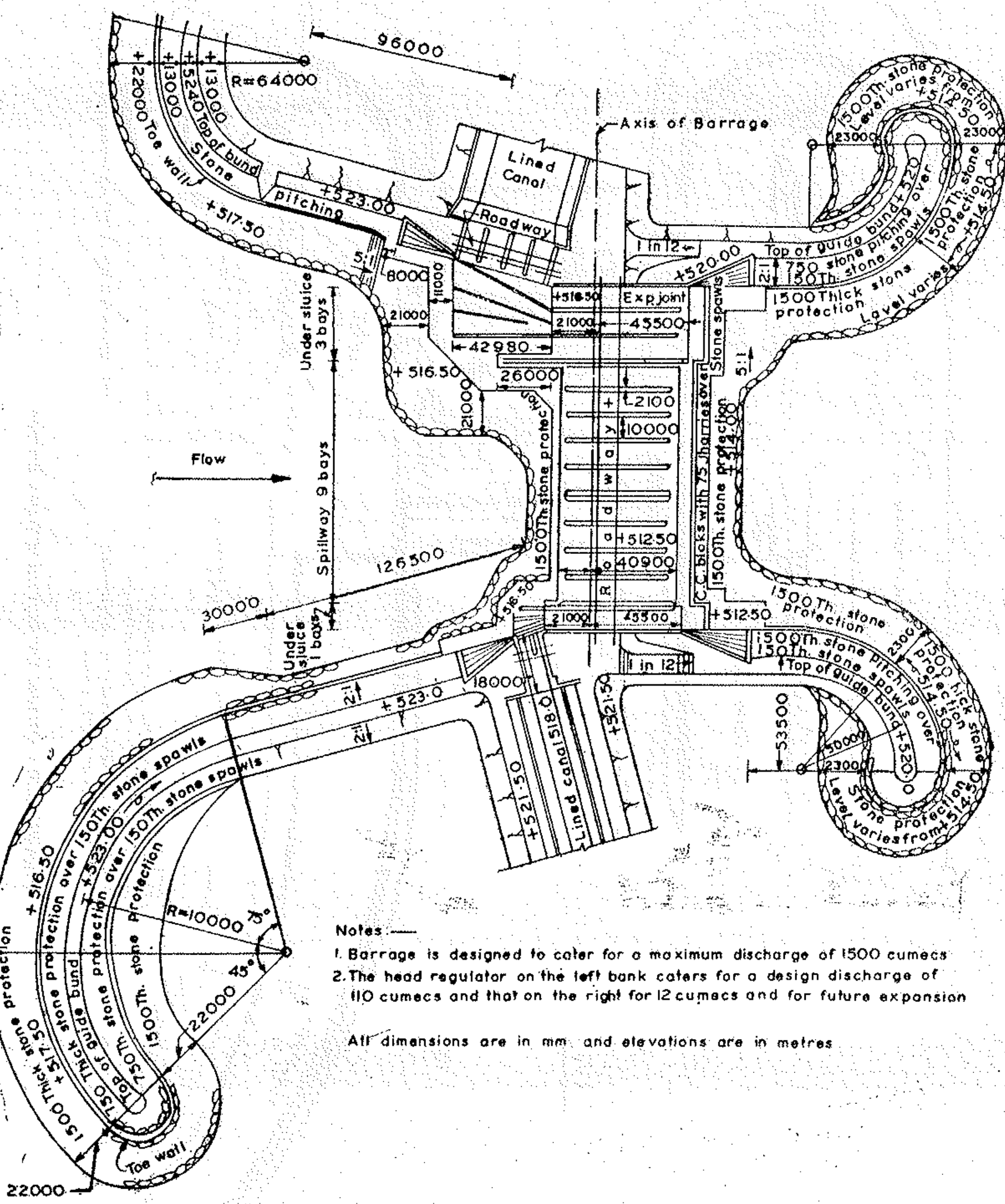


FIGURE 5.19 : Detailed Layout Plan of Khanabed Barrage.

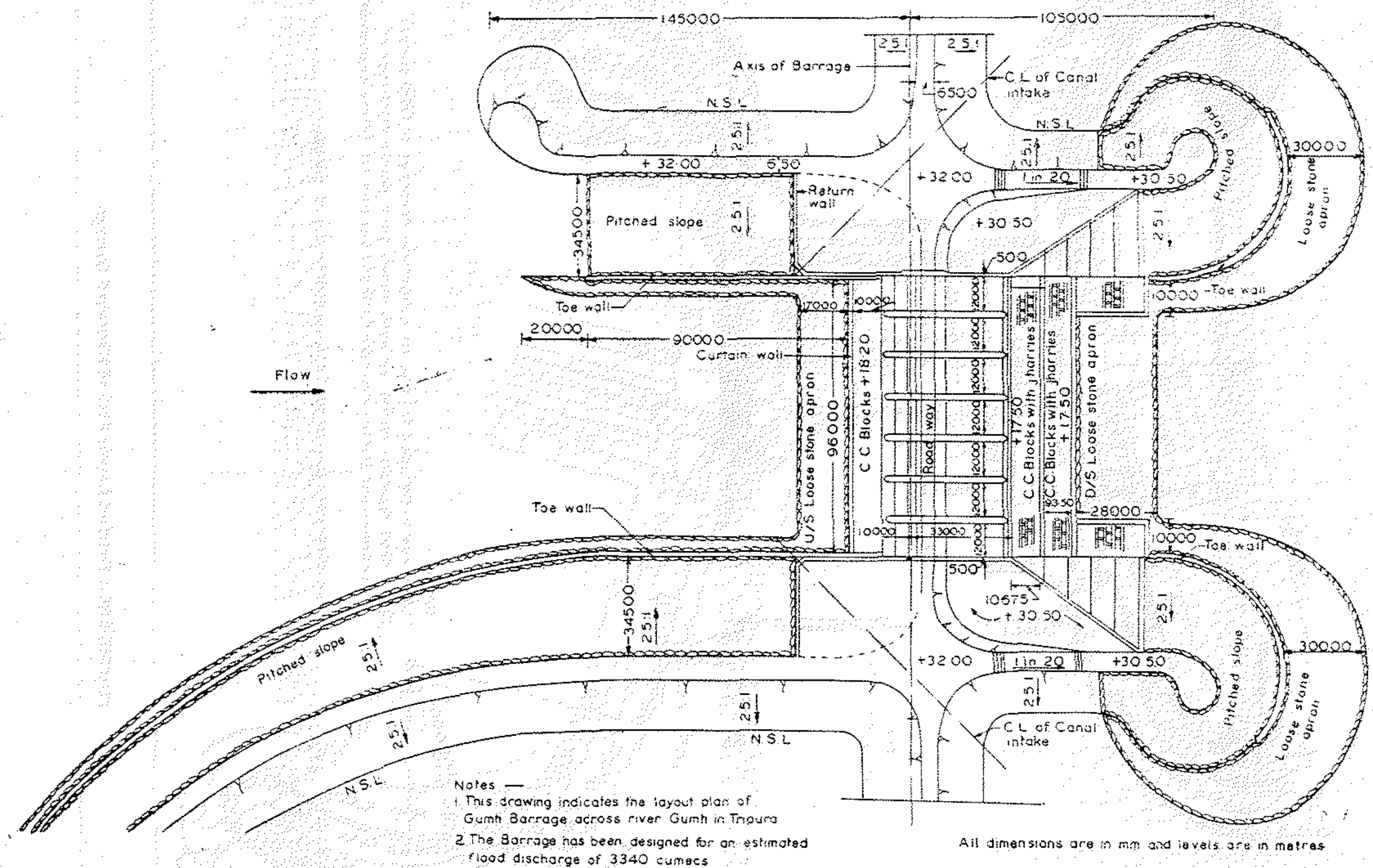


FIGURE 5.20 : Detailed Layout Plan of Gumti Errrage.

5.6.6. *Design Values to be Checked*

5.6.6.1. Sliding

In case clayey strata is obtained below permeable (sandy) strata at a small depth, the sliding effect at that elevation should also be checked. It is preferable to use a lower value of cohesive strength 'C' obtained from the laboratory analysis.

5.6.6.2. Bearing Pressure

The design load reckoning uplift pressure into account transmitted by the structure and also neglecting uplift pressure may be calculated and compared with the permissible bearing capacity of the clay formation.

5.6.6.3. Settlement

During various stages of construction and maintenance, the load on the floor will be different. Maximum loading occurs at a time when the entire pit is dewatered and piers are raised to the full height during the construction stage. During operation conditions because of the induction of uplift pressure below the floor, the net load normally gets reduced. Generally, settlement calculations may be done for this load transmitted at the bottom of the floor. As the samples are generally collected at intervals of about 5 metre depths along the bore hole, the settlement computations are made accordingly assuming the soil to be of uniform character along a depth of 5 metres. Since the top layers of clay strata are generally removed if sandy strata is met with at shallow depths before again meeting with clayey strata below the sandy strata, in those cases, the top clayey strata is ignored in the settlement computations.

Apart from the total settlement, implications of differential settlement have to be kept in view. It would be desirable to induce as much consolidation as is practicable before the structure is built so that the total settlement likely to occur later on over a period of time can be correspondingly minimised. This, in tern, will also reduce the differential settlement which is more important, especially for a reinforced concrete raft.

5.6.7 There is a general feeling that gravity type of floor is desirable instead of raft type in clayed strata due to settlement problems. Either of the two types can be adopted subject to economic considerations. For design of raft, guidance may be taken from IS : 2950 (Part I)—1973—Code of Practice for Design and Construction of Raft Foundation. The length of the floor is determined from surface floor considerations since exit gradient in clayey soil has no relevance.

5.6.8. *Arrangement of cut-off sheetpile*

5.6.8.1. Upstream

It is desirable to have the bottom of the pile either embedded into the clay layer or it should be at least one metre above the clay layer to prevent hetrogeneous conditions of seepage (For this purpose, continuous probing of the clayey strata along this line will be necessary at the time of actual driving of piles). In case the pile line is embedded into the clay layer, then the seepage flow under the raft is considerably reduced resulting in reduced uplift pressure. In case the sheet pile bottom is kept at one metre above the clay layer, then there will be seepage flow through the sand strata. It is likely that the clay layer may not be at a uniform elevation in which case, the sheet pile depth may have to be varied if one metre clearance between the clayey strata and sheet pile bottom is to be obtained. This may be difficult to achieve in the field condition. In case the piles are to end in clayey strata, no such problem will exist and it can be conveniently driven to the design depth or slightly more to intercept the clay layer as the case may be. The latter arrangement provides depths more than envisaged and hence preferable.

5.6.8.2. Downstream

In the case of downstream sheet pile, it is not a desirable feature that the bottom of the sheet pile intercepts the clay layer as this will lock up the seepage flow and thereby build up uplift pressures under the raft. From this consideration, it may be necessary to raise the downstream sheet pile elevation at least by a metre. The criteria of exit gradient may not be relevant in respect of a condition where the upstream sheet pile is anchored in the clay and the downstream pile line is above the sandy layer.

5.6.8.3. Boxing

In some cases, it may so happen that the clay layer may be at a high level that while it may be possible to anchor the upstream pile line into the clay, lifting the downstream pile above the clay layer may not be feasible. In those cases, it may be necessary to take both the sheet pile lines to the design depth and the foundation may be boxed up. Though the foundation may be completely boxed with sheet piles, nevertheless, there will be some seepage ; but this would not be amenable to precise assessment.

The implications of lesser seepage on the possible settlement need to be reviewed in such cases. The vertical load occurring on the foundation strata below the raft would be increased and hence the settlement characteristics will be aggravated. From the consideration of settlement, boxing of the entire area with the sheet pile in certain cases may not be a desirable feature.

An alternative arrangement would be to have window slits in the upstream sheet pile line. These openings would provide connection from the upstream pool to the Boxed foundation strata thereby introducing uplift pressures on the raft. In order to prevent building up of 100 percent uplift pressure by lockage, it would be necessary to provide some pressure release arrangement for the boxed foundation strata to the downstream end. Providing of windows on the downstream pile is considered not desirable since any exposure of pile line due to scour may pose a problem of removal of fine material from the boxed foundation strata. The conventional pressure release arrangement just at the junction of the glacis with the downstream floor with exit points in the pier having non-return flap valves may be practicable with a view to release the locked up pressures from the boxed foundation. In such cases, it would be necessary to have piezometric observations at two sections in each unit of the raft.

5.6.9. *Abutment and Return wall Design*

In view of the possible settlements occurring under the clay strata and to prevent differential settlement between the abutment and the raft, it would be desirable to design the abutment and the portion of the end bay raft as a continuous structure so that the problem could be obviated. As regards the return walls, these could be designed as supported on well foundations. For design of well foundation under piers and abutments, guidance may be taken from the following publication and also other standard text books on well foundation :

"Paper No. 238 Considerations in the Design and Sinking of well Foundations for Bridge Piers-B. Balwant Rao and C. Muthuswamy—Journal of the Indian Road Congress Vol. XXVII—3, August, 1963 and Vol. XXVII—5, July, 1965".

5.6.10. *Location of Double Piers*

In view of the different types of foundation strata encountered over the full or part length of the barrage, it may become necessary to limit the length of the units of the barrage with double piers at such places where there is marked change in the foundation strata.

CHAPTER 6

Hydraulic Designs

6.0. Introduction

The designs of the diversion structures are to be carried out in two parts, namely hydraulic and structural. In the hydraulic designs, overall dimensions and profiles of the main structure and a few of the components are worked out so that satisfactory hydraulic performance of the structure can be ensured. In the structural designs, the various sections and reinforcement details wherever needed are worked out. The dimensions fixed up by the hydraulic designs are again got tested by model tests for their adequacy and after they are finalised based on the results of the model tests, structural designs are takgn up. Details are then worked out to have a structure which will be safer under any possible and probable combination of loadings. In both the cases, the diversion structure has to be properly designed for both the surface-and sub-surface flow conditions. The surface designs will include the fixing up of waterway, top profile of various structures energy dissipation arrangements, protection works, scour values, length and protection of divide walls, alignment, levels and protection of guide bunds, afflux bunds, etc. The sub-surface designs will include fixing of the depth and section of cut-offs, uplift pressure calculations, exit gradient, etc. The theories involved in the computation of these and guide lines for their designs have all been discussed in this chapter. Typical calculations for various items are presented in Appendix 6.1.

6.1. Development of Weir Design

The principes of design of barrages and weirs have been evolved over a number of years as a result of analysis of various failures of the diversion structure constructed. The failures can be attributed to four main causes, acting singly or in combination. They are (*i*) undermining through piping due to excessive exit gradient, (*ii*) eruption of floor caused by uplift exceeding gravity forces, (*iii*) deep scour immediately upstream and/or downstream of the pucca floor and (*iv*) miscellaneous causes like faulty regulation, faulty constrution, etc.

6.1.1. Failures of Khanki and Narora weirs and Col. Clibborn's experiment.

6.1.1.1. The law of flow of water through permeable soil was enunciated for the first time by Darcy, who as a result of experiments stated that the velocity varied cirectly as the head and inversely as the length of the path of flow.

Hence $V=\frac{Kh}{l}$

where h=head, l=length of seepage path and K=a constant.

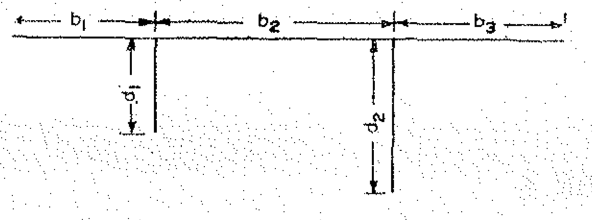
The law is the same as the law for movement of water in capillaries enunciated by Poiseuille in 1841-42.

6.1.1.2. Experiments were conducted by Col. Clibborn in Roorkee during 1895—97 to test the validity of this law. The relation obtained from his experiments between velocity head and length of flow was in keeping with Darcy except that at high heads slight departures were noticed. The experimental results were then checked on the prototype at Narora weir. Unfoıtunately two days after the experiments, there was a failure of the weir due to excessive uplift pressures.

6.1.2. *Bligh's Theory*

The failures at Narora, Khanki and Col. Clibborn's experiments led to the hydraulic gradient theory for weir design by Sir John Ottley and Thomas Higham. In 1910, Bligh perfected the hydraulic gradient theory which later on came to be universally known as Bligh's Theory.

6.1.2.1. Bligh assumes that the hydraulic slope or gradient is constant throughout the length of creep. (Figure 6.1.) 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" 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 under-



The length of travel in a weir profile as given in the figure would be $L = b_1 + d_1 + d_1 + b_2 + d_2 + d_2 + b_3$
$= b_1 + 2d_1 + b_2 + 2d_2 + b_3$
If H = total head over the weir, the loss of head per unit length of creep would be

$$\frac{1}{C} = \frac{H}{b_1 + 2d_1 + b_2 + 2d_2 + b_3}$$

FIGURE 6.1 : Explanatory figure for Blighs Theory

mining if the ratio $L/H = C$ is not less than a certain value given in Table 6.1. below.

TABLE 6.1

Minimum safe values of Bligh's 'C'.

For fine micaceous sand in North Indian Rivers	C=15
For coarse grained sand	C=12
For sand mixed with boulder and gravel and for loam soil	C=5 to 9

The second condition of equilibrium is that the weight of the apron must be sufficient to counter-balance the upward pressure.

6.1.2.2. Bligh stated that the length of the path of flow had the same effectiveness ; length for length in reducing uplift pressures whether it was along the horizontal or the vertical. From the meagre data available at that time, Bligh evolved a simple formula which cannot be applied in all conditions. Following Buckley, Bligh assumed the percolation water to "creep" along the contact of the base profile of the weir with the sub-soil, losing head en-route, proportional to the length of its travel.

6.1.2.3. The length of travel in a weir profile as given in Figure 6.1 would be $L = b_1 + d_1 + d_1 + b_2 + d_2 + d_2 + b_3 = b_1 + 2d_1 + b_2 + 2d_2 + b_3$. If H=total head over the weir, the loss of head per unit length of creep would be

$$\frac{1}{C} = \frac{H}{b_1 + 2d_1 + b_2 + 2d_2 + b_3}$$

This loss of head per unit length is the average hydraulic gradient and C is called the percolation coefficient. If in a design C were less than the safe assigned value for the given sub-soil, the design was considered safe.

6.1.2.4. Due to its simplicity, Bligh's theory found general acceptance. Some works designed on the basis of Bligh's theory failed while others stood, depending on the extent to which they ignored or took note of the importance of vertical cut offs at the upstream and downstream ends.

6.1.2.5. The general criticism of Bligh's method is (*i*) that the slope (Hydraulic Gradient) is considered to remain constant over the entire length of the line of creep. Actually, as the seepage theory developed now shows, this gradient is varying widely at different points of the seepage path, (*ii*) it does not give any weightage for creep along vertical lines and (*iii*) it assumes the flow to occur along the line of contact.

6 .1.2.6. However, Bligh's theory enjoyed wide recognition due to fundamental assumption of the existence of harmless limit for the percolation velocity and also on account of the simplified assumption of a sloping uplift diagram.

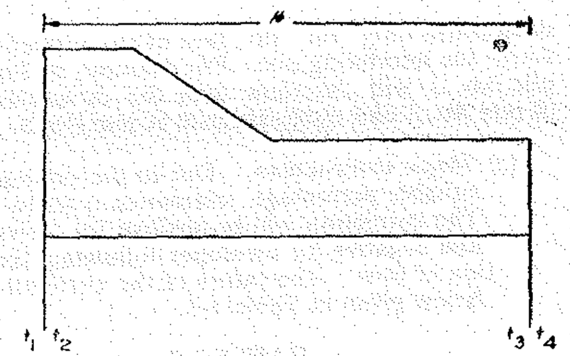
6.1.3. *Lane's Theory*

During the next 30 years, it was gradually recognised that vertical sections of the line of creep contributes more towards reducing the danger of piping than horizontal sections of equal length. The difference is due to the nature of the flow net and would be there even for homogeneous soils. This is further accentuated by the fact that the sub-soil of dams is commonly of sedimentary origin and sedimentary deposits are always much less permeable in the vertical direction than in the horizontal direction. If K_h and K_v are respectively the coefficients of permeability in the horizontal and vertical directions, the loss in head per unit of length of vertical sections of the line of creep is roughly equal to the ratio K_h/K_v times that of horizontal sections. The value of the ratio ranges from 2 or 3 to almost infinity in exceptional cases, depending on the details of stratification and the importance of the variations of the permeability in the vertical direction.

6.1.3.1. To take account of the greater efficiency of vertical sections of the line of creep, Lane modified the original Bligh's procedure by the assumption that every horizontal section of the line of creep was only one third as effective as a vertical section of the same length. With reference to Figure 6.2 on this assumption the equation,

$$C_w = \frac{\frac{1}{3} B + (t_1 + t_2 + t_3 + t_4 \text{ etc.})}{h_{or}}$$

was obtained. The value C_w is known as Lane's weighted creep ratio. Lane's theory is based on his review of the various works which stood the test of time and also those that had failed. Based on an analysis of about 280 dam foundations of which 24 had failed, the values of weighted creep ratio C_w were suggested as given in Table 6.2.



Lane's weighted creep ratio $CW = \dfrac{\frac{1}{3}B + (t_1 + t_2 + t_3 + t_4 \text{ etc})}{h_{cr}}$

h_{cr} = The smallest head at which piping ever occured

FIGURE 6.2 : Explanatory figure for Lane's Theory.

TABLE 6.2

Maximum values of Lane's weighted creep Ratio

Nature of foundation material	C_w
Very fine sand	8.5
Fine sand	7.0
Medium sand	6.0
Coarse sand	5.0
Fine gravel	4.0
Medium gravel	3.5
Coarse gravel	3.0
Boulders and gravel	2.5

6.1.3.2. Lane's weighted creep approach is an improvement on Bligh, but still it is purely empirical and leads to a design with an unknown factor of safety. The values of C_w given in Table 6.2 above represent maximum rather than average value and the values of h_{cr} obtained by means of the above given equation represent the smallest heads at which piping ever occurred. According to Lane, water may occasionally travel along the line of creep, because it is difficult to secure intimate contact between the flat surface of solid foundation and granular soil upon which it rests. If there is indeed lack of or poor contact, then water percolating along the line of creep will meet with tess resistance than that which travels through undisturbed soil.

6.1.3.3. Lane's theory is still popular in the USA and less important dams of USSR are designed on its basis. For important cases, results obtained by mathematical approach of subsoil flow are accepted all over.

6.1.3.4. *Appraisal of Lane's theory.* Lane's theory represents an empirical approach based on long experience and one should be able to make a safe design on the basis of Lane's theory, if he is fully familiar with its limitations. Care should, however, be taken to provide cut off to take care of the scour at the d/s end and also to provide necessary filter layer to take care of the unrelieved gradient at the tail. The factor of safety of works designed on the basis of Lane's theory would be quite uncertain and the design in some cases whould be quite wasterful.

6.1.4. *Modern Approach to Design of Wiers and Barrages*

The modern approach to the theory of weir design is that (*i*) seepage takes place according to the theory of seepage flow throughtout the underling strata and (*ii*) that the stability of granular particles depends on the limiting value of hydraulic gradient at the upper surface of the granular material which gradient to satisfy the consideration of equilibrium is to be smaller than the critical gradient. The second principle was enunciated by Terzaghi in 1925 and discovered independently by Khosla a few years later.

6.1.5. *Khosla's Theory*

The distribution of uplift pressure is not linear. This, of course, follows from the fact that seepage take place according to the theory of seepage flow and is governed by the Laplacian equation and appropriate boundary conditions. If the Laplacian equation could be 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 shape of the base of the foundations presented by an actual work.

6.1.5.1. 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. Mathematical solutions of flow nets have been obtained for a number of such simple standard firms, the most useful of which are : (*i*) A straight horizontal floor of negligible thickness with a sheet pile line at either end (*ii*) a straight horizontal floor depressed below the bed but with no vertical cut off and (*iii*) a straight horizontal floor of negligible thickness with a sheet pile line at some intermediate position.

6.1.5.2. The results of the mathematical solutions of these forms have been discussed in publication No. 12 of the Central Board of Irrigation and Power titled "Design of Weirs on Permeable Foundations" by Rai Bahadur A.N. Khosla, Dr. N.K. Bose and Dr. E. Mckenzie Taylor. The solutions are given in the forms of curves (Figure 6.3) from which the percentage pressures of the proportion of the residual head to the total seepage head can be determined at key points. The pressures at intermediate points are assumed to vary along a straight line, which does not introduce any appreciable error.

6.1.5.3. Dr. Khosla has shown that the percentage pressures observed from the curves for the simple for s into which the profile is broken up can hold good for the assembled profile as a whole subject to certain corrections as given below :

(*a*) **Correction for Thickness of Floor :** In the standard forms with vertical cut-off, the thickness of the floor is assumed to be negligible and hence the value observed from the curves, refer to the top level of the floor. However, due to the thickness of the floor provided, corrections are to be applied for the points at the bottom of the raft and these are interpolated by assuming straight line variation. The correction may be positive or negative according to the points of observation.

(*b*) **Correction for Mutual Interference of Sheet piles.** The correction to be applied due to the mutual interference of sheet piles is given by the formula.

$$C=\frac{D}{b'}\times\frac{d+D}{b} \text{ where}$$

C—is the correction to be applied as percentage of head.

b'—is the distance between the two pile lines (See Figure 6.4)

D—the depth of the pile line, the influence of which has to be determined on the neighbouring pile of depth d, D is to be measured below the level at which interference is desired.

d—the depth of pile on which the effect of pile of depth D is sought to be determined and

b—total floor length.

The correction is positive for points in the rear or backwater and subtractive for points forward in the direction of flow. This correction equation does not apply to the effect of an outer pile on an intermediate pile 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. The effect of interference of a pile is to be determined only for the face of the adjacent pile towords the interfering pile. For example, in Figure 6.4, pile No. 2 will interfere with the downstream face of pile No. 1 and upstream face of pile No. 3.

(*c*) **Slope Correction.** Due to the sloping floor, a suitable percentage correction is to be applied. The correction will be positive for down slopes and negative for up-slopes following the direction of flow The values of the correction are as below given in Table 6.3.

TABLE 6.3

Values of Slope Correction

Slope (V : H)	Correction of % of pressure
1 : 1	11.2
1 : 2	6.5
1 : 3	4.5
1 : 4	3.3
1 : 5	2.8
1 : 6	2.5
1 : 7	2.3
1 : 8	2.0

The correction is applicable to the key point of the pile line fixed at the beginning or the end of the slope. Referring to Figure 6.4, the correction is applicable only to point E of pile No. 2. The percentage correction given by Table 6.3, is further to be multiplied by the proportion of the horizontal length of slope to the distance between the two pile lines in between which the sloping floor is located.

6.1.5.4. *Exit Gradient.* Another highlight of Khosla's experiments and analysis is that the criteria for safety against piping is not the average hydraulic gradient as enunciated by Bligh but the floatation gradient or in other words, the pressure gradient at the exit.

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

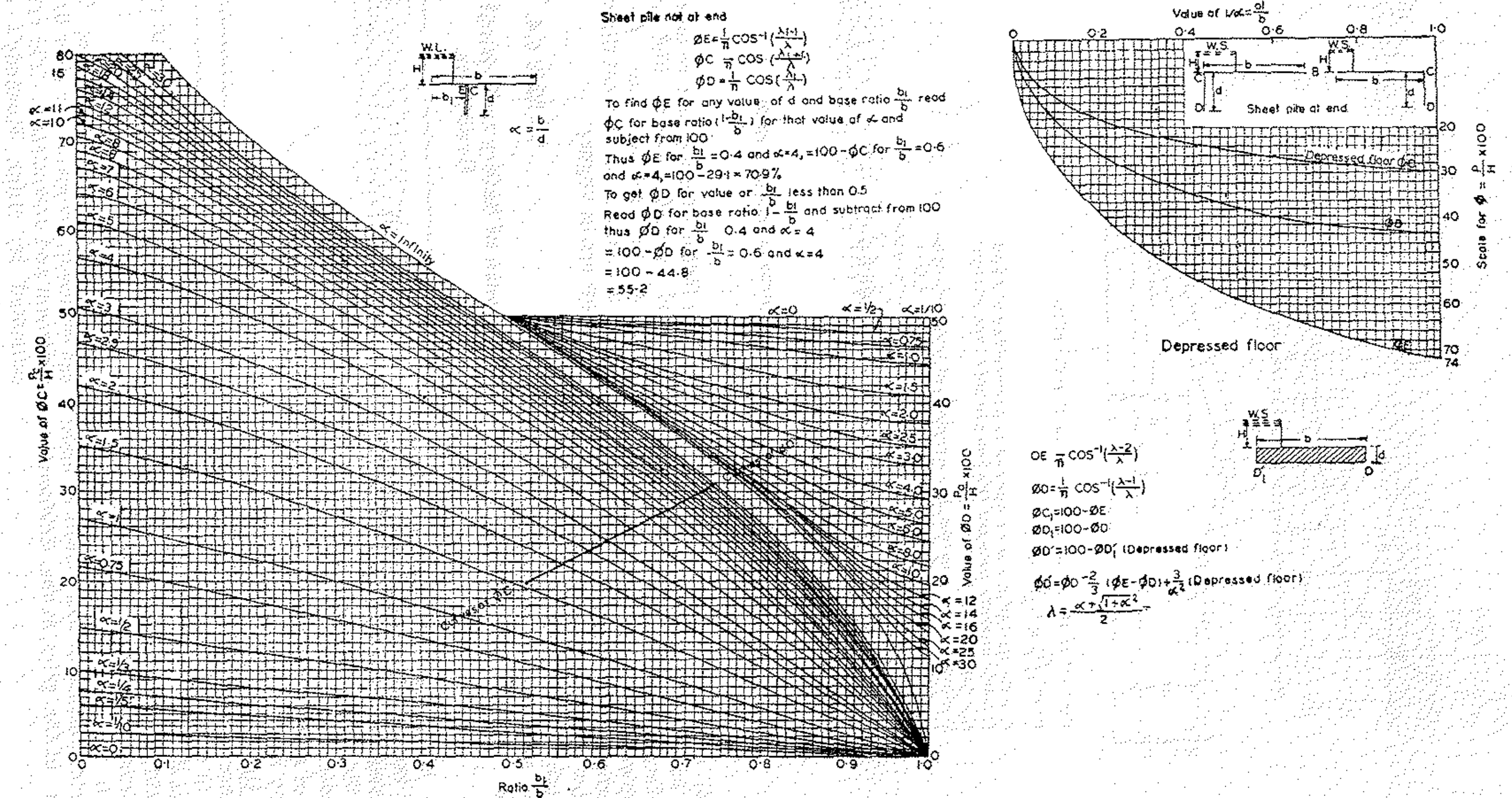
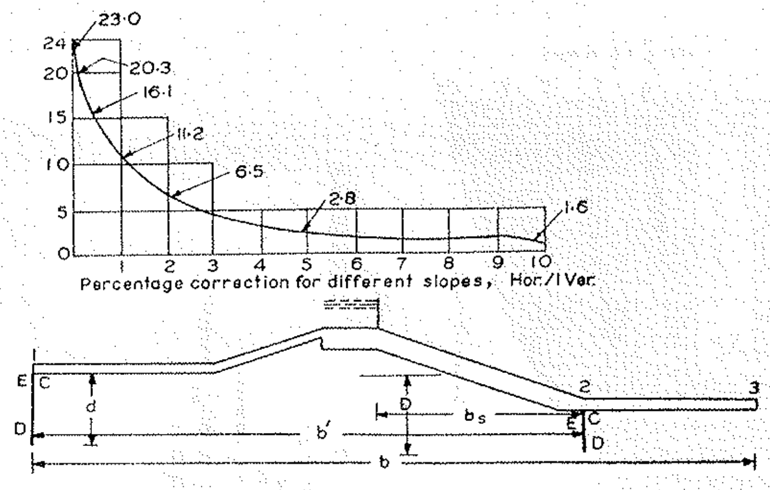


FIGURE 6.3 : Khosla's Theory Curves for Estimation of Uplift.



Correction for mutual interference of sheet piles

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

C is the correction to be applied as percentage of head

b' is the distance between the two pile lines (see sketch)

b_s = Sloping length of floor

D = Depth of the pile line, the influence of which has to be determined on the neighbouring pile of depth d, D is to be measured below the level at which interference is desired

d = Depth of pile on which the effect of pile of depth D is sought to be determined

b = Total floor length

FIGURE 6.4 : Explanatory figures for corrections in Khosla's uplift pressure values.

latter. The frictional resistance, cohesion, etc., of the adjacent soil will have to be considered in certain cases. Once the surface particles are disturbed, the resistance against upward pressure of water will be further reduced, tending to progressive disruption of the sub-soil. The flow gathers into a series of pipes in the latter and dislocation of particles is accelerated. The sub-soil is thus progressively undermined. Soil erosion can also occur through natural pipes or faults in the sub-soil.

It is not enough to design a structure so that the exit gradient at the tail end does not exceed the critical value of the gradient. Even a slight increase in this value will upset the stability of the sub-soil at the exit end. This indicates the need for the application of a generous factor of safety in the designs. The various uncertainities may be (*i*) non-homogeneity of the foundation soil even though definite stratification may be absent (*ii*) difference in the packing and pore space (*iii*) local intrusions of clay bed or very porous materal (*iv*) faults and fissures in the sub-soil formation (*v*) presence of continuous clay beds or bands needing special treatment, etc. The factor of safety to be applied would take care of all these uncertainties besides covering cases where due to retrogression of bed levels or flood scour, the upper part of the piles at the downstream end is exposed.

The factor of safety has to take note of the class of material, specific weight and pore space, angle of repose of the sub-soil, etc. As not much correlated data is available, at present the values given in Table 6.4 below are in vogue.

TABLE 6.4.

Factors of Safety to Critical Values of Exit Gradients.

Sub-soil	Factor of safety
Shingle	4 to 5
Coarse sand	5 to 6
Fine sand	6 to 7

These values will not apply to clays which are more or less impervious.

6.1.5.5. *Scour and Protections* Due to the passage of floods, scours of the river bed occur both on the upstream and downstream. The extent of scour can be estimated by using Kennedy's or Lacey's formula.

Since Lacey's formula gives somewhat higher values than Kennedy's, it is accepted as being on the safer side. The normal depth of scour '*R*' below the High Flood Level depends on the intensity of discharge '*q*' and the value of silt factor '*f*'. Lacey's depth of scour '*R*' is applicable to regime flow only. Where there is likelihood of disturbance due to curved flow or otherwise, the depth of scour would be more. According to Lacey the scour can be grouped under four classes as given in Table 6.5.

TABLE 6.5

Classes of Scour.

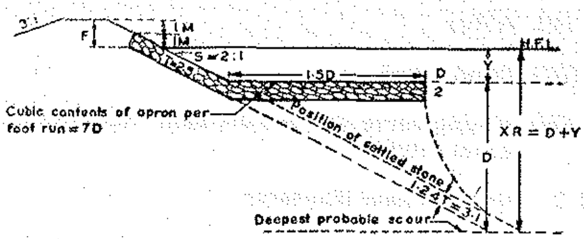
Class	Reach	Depth of Scour
A	Straight	1.25 R
B	Moderate bend	1.50 R
C	Severe bend	1.75 R
D	Right angled bend	2.00 R

Class A Scour is likely to occur anywhere just below the loose aprons. Class B scour is likely to occur anywhere along the aprons of guide banks in the straight reach and classes C and D at and below the noses of guide banks, or at the loose weir aprons, should heavy swirls set in for any reason.

Upstream and Downstream cut offs are taken down to depths such that they provide safety against certain values of estimated scour which are discussed further in detail in Para 6.9

In addition to the cut-offs, pervious protections to the pucca floor are also to be provided both on the upstream and downstream sides beyond the floor. These are provided in the form of cement concrete blocks and loose stone apron. 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 the protections are damaged they should be made good at once. Their existence will be a definite safeguard against any damage to pucca floor.

The loose stone protection, also called the launching apron, beyond the block protection, keeps the scour holes at a safe distance from the block protection and the pavements. The slope of the scour hole is assumed to be generally not steeper than 2 : 1 and not flatter than 3 : 1 and for rivers with very fine sand or silty bed, the slope could be taken as 3 : 1 and for rivers in bouldery reaches, the slopes can go upto 1.5 : 1 in exceptional cases. According to spring, the quantity of loose stone apron should be sufficient to afford (approximately about 1 m) cover over the slope below the level at which the apron is originally laid to the bottom of the deepest scour that is likely to occur at the particular locality. Referring to Figure 6.5, the depth of covering over the slope made by the



Reffer chapter 2- Definition and notations
Para 6.1.5.5
Table No. 6.6

Legend :—
F = Free board
XR = Scour depth below HFL
Y = Depth of water above bed at high flood

FIGURE 6.5 : Explanatory Figure for Calculation of loose stone apron.

falling apron due to scour should be 1.25 *T* where *T* is the thickness of stone on the slope. The values of *T* is generally taken from the values given in Table 6.6 below.

TABLE 6.6

Thickness (mm) of Stone Pitching necessary to Protect Sand Surfaces.

Types of River bed materials	River slop in m/km 0.05	0.15	0.2	0.3	0.4
Very Coarse	400	500	550	650	700
Coarse	550	650	700	800	850
Medium	700	800	850	950	1000
Fine	850	950	1000	1100	1150
Very Fine	1000	1100	1150	1250	1300

The estimation of scour and length and thickness of protections are discussed in more details in Para 6.9.4.

With the above discussed background material, guidelines for fixing the various parameters under Hydraulic Designs are given in the succeeding paras.

6.2. Basic Parameters

As already mentioned in Chapter 5 on Planning and Layout, the following basic parameters are to be decided upon before taking up detailed hydraulic designs :

(*i*) Design flood discharge

(*ii*) Afflux

(*iii*) Pond Level

(*iv*) Rating curve and downstream retrogression of water levels.

6.1.2. *Design Flood Discharge*

The maximum flood discharge in the rivers may be worked out by any one or more of the following methods and formulae taking into account the variations in rainfall, type of catchment, available records both within the catchment and in the region :

(*i*) Unit Hydrograph method

(*ii*) Frequency analysis

(*iii*) Slop Area Method

(*iv*) Use of empirical formulae such as

(*a*) Dicken's

(*b*) Ryve's

(*c*) Ingli's

(*d*) Nawab Jung Bahadur's

(*e*) Creager's

(*f*) Fuller's

(*v*) Use of flood curves such as

(*a*) Beale's and Whiting's

(*b*) Curves of the Institution of Civil Engineers (London).

For details on the various methods, standard books on Hydrology or CWC Publication on 'Estimation of Design Flood' may be referred.

6.2.2. *Afflux*

As already mentioned in Para 5.1.2., it is very important to decide about the limiting values of afflux to proceed with the deigns. The afflux is generally limited to about 1 m for structures on alluvial rivers at highers reaches. In lower reaches it is limited to about 0.3 m depending on the nearness to important towns, complexes, etc. In very steep reaches of the rivers with boulders of rocky bed, the afflux may safely be higher, say of the order of 2-3 metres. However, this has to be determined from considerations of economy. Refer Table 5.1 also.

6.2.3. *Pond Level*

From the various considerations mentioned in Para 5.1.4., the Pond level to be adopted in the designs is fixed up. The Full Supply Level of the Canal at the head will be fixed on the L-Section of the canal taking off-from the diversion structure. By adding about 0.5 to 1.0 m to this F.S.L. (depending upon specific requirements in each case) to provide a working head through the canal head regulator, the Pond level is obtained. This extra head will usually take care of the losses through the head regulator and the provision for silting of the canal in the head reaches and consequent increase in the F.S.L. of the canal.

6.2 4. *Rating Curve and Downstream Retrogression Water levels*

In the design of the diversion structure, the rating curve, otherwise called the Gauge-Discharge Curve, plays a very important part. Hence it is very imperative to have a good rating curve of the river at the site of the structure. As already mentioned in Para 5.1.1, a statistically analysed Gauge-Discharge curve must be prepared. If no data is available on the Gauge-Discharge values, the same may be worked out by Manning's formula. This is the unretrogressed G.D. curve to be used for waterway and free-board calculations.

As a result of ponding up of rivers flow, relatively silt-free water escaping over the diversion structure pick up bed silt downstream of the structure and progressive degradation or retrogession of levels takes place. In the first few years the retrogression of levels is rapid and progressive. It may even range between 1.25 m to 2.25 m. This lowering of bed levels in the early stages after commissioning of the structure, if not duly allowed for in the designs, may result in its failure. The structure may be undermined by an increase in the value of the exist gradient beyond safe limit. The energy dissipation through hydraulic jump may be impaired due to lowering of downstream water levels causing damages to the structure and the protection works. As a result of the retrogression in bed levels, while the low water levels have been found to drops from about 1.25 to 2.25 m, the maximum flood levels have not been known to have dropped by more than 0.3 to 0.5 m.

The process of recovery of downstream bed levels after the initial retrogression is slow but steady, taking even 20 to 30 years The bed levels may even rise above the original levels. Hence while fixing up the retrogression, the following points are to be borne in mind :

(*i*) There is more retrogression in rivers carrying more silt

(*ii*) The finer the bed material, the more is the possibility of retrogression.

(*iii*) The steeper the slope of the river, the greater is the retrogression.

A retrogressed rating curve to be used in the designs for energy dissipation, protection works, etc. may be obtained from the normal rating curve (unretrogressed) by subtracting the retrogression for various flood discharge which can be taken to vary in a logarithmic fashion between the design flood and low flood values. See Figure 5.1. for a typical Gauge-Discharge Curve.

6.3. Waterway

As already discussed in Para 5.1.5., the wateı way of the diversion structure has to be carefully planned. The waterway includes that of undersluice, spillway and riversluice bays if any. In deep and confined rivers with stable banks the overall waterway between the abutments would normally be adjusted to the actual width of the rivers at design flood level. For shallow and meandering alluvial rivers, the minimum stable width can P be calculated from Lacey's formula, $P=4.83$ $Q^{1/2}$ where Q is in cumecs. Where the width of the river is very large, the width of the diversion structure is limited to Lacey's width × Looseness factor desired and the balance width is blocked by Tie bunds with suitable river training measures.

6.3.1. With the selection of the span width of each bay and adjustment of the number of spillway, undersluice, riversluice bays and also fish pass if any, the width between the abutments can be calculated by adding the thickness of single and double piers in between. This would give an idea of the lossesness factor to be adopted in the designs and the likely effects thereof.

6.3.2. The crest levels of the different bays may be fixed up in light of the discussions in Para 5.1.6. With these tentative values, the adequacy of the waterway for passing the design flood within the permissible afflux needs to be checked up. Otherwise, the waterway and crest levels will need to be readjusted in such a way that the permissible values of afflux is not exceeded.

6.3.3. Discharge $Q=C.L.H.^{3/2}$ where, L is the clear waterway, H is the total head over creast and C is the coefficient of discharge.

6.3.3.1. Total head H includes the head of water on the upstream side over the crest and the head due to velocity of approach. For calculating the velocity of approach, the discharge per unit width of the diversion structure (total width) is divided by the scour depth R below HFL calculated by the following formulae :

$$R=0.475\left(\frac{Q}{f}\right)^{1/3}$$ when looseness factor is more than 1.

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

when looseness factor is less than 1.

where f= Silt factor to be estimated by knowing the average particle size m_r in mm of soil.

$$f=1.76\sqrt{m_r}=290\ (R^{1/2}\ S)^{2/3}$$

q= intensity of flood discharge.

6.3.3.2. The coefficient of discharge C depends on the drowning ratio obtained with different water levels assumed on the upstream side and estimated from the unretrogressed Gauge-Discharge curve.

$$\text{Drowning Ratio}=\frac{\text{D/S W.L.—Crest Level}}{\text{U/S W.L.—Crest Level}}$$

It may be noted that the upstream water level is initially assumed by adding afflux to the downstream water level.

The coefficient of discharge will need to be estimated or determined from model experiments. In the recent past, some experiments have been conducted by different research stations and curves have been prepared correlating the drowning ratio with the coefficient of discharge. These are presented in Figure 6.6. While the conditions and parameters under which these experiments have been carried out are not known, it has been observed that these curves give fairly satisfactory values of coefficient of discharge to be used in the calculations which can be later verified by model experiments and modified wherever necessary.

6.3.3.3. Substituting the various values of C, L and H for the initially assumed value of upstream water level and the waterway, the value of discharge that can be passed through the structure can be worked out. If it is equal to or nearly equal to the design discharge, the assumptions are okey ; otherwise, the procedure may be repeated to get a near value.

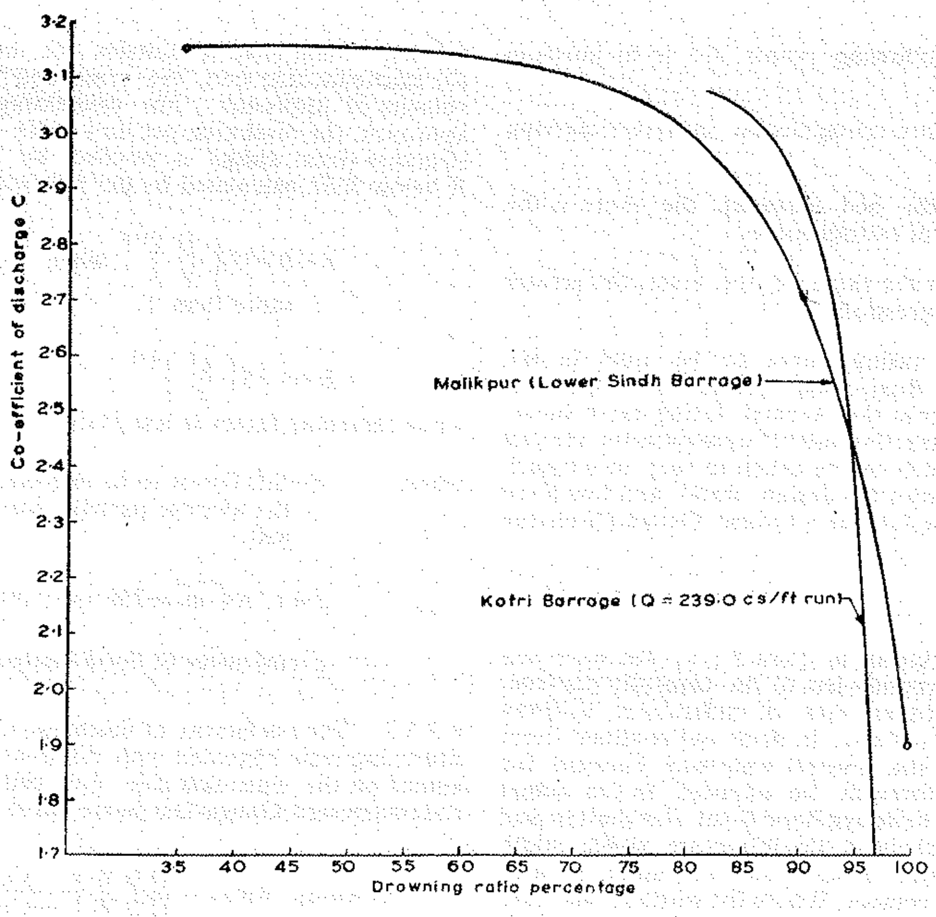


FIGURE 6.6 : Curves for Estimation of Co-efficient of Discharge Vs. Drowing Ratio.

6.3.3.4. Since the crest levels of spillway, undersluice and rivers-sluice bays would be different, the discharges passing through each will have to be estimated separately and totalled up. Wherever silt excluder tunnels are proposed to be provided in the undersluice bays, the discharges through these tunnels and over them will need to be calculated separately and added up. For calculation of the discharge through the silt excluder tunnels, it may be treated as a large vertical orifice and the appropriate formula for free flow or drowned condition or partially drowned condition may be used. In some small structures with a combination of weir and sluice bays, sometimes breast walls are provided in the sluice bays to reduce the height of the gates. In those cases also, for estimation of discharges through the sluice bays, the appropriate formula of large vertical orifice may be used.

6.3.4. From the above, it will be clear that the waterway, crest levels and afflux are all inter-linked and hence various combinations of these can be had for passing the sarre design discharge. The final features to be adopted in the designs should be nocsistent with economy and safety.

6.3.5. Typical waterway calculations are presented in Appendix 6.1.

6.4. Energy Dissipation

The standing wave is considered as one of the most effective means of killing the surplus energy in hydraulic structures. The elevation and length of the downstream cistern or pucca floor depends on the position and height of the standing wave under various sets of conditions. The formation of standing wave depends on the discharge per unit width, afflux, shape of the glacis, etc. The following points are to be carefully noted in the designs for energy dissipation :

(*i*) On a level floor with low friction, the position of the standing wave is unstable. For a slight change in the depth or velocity, the position of the standing-wave will vary through a wide range. The position of the jump on a horizontal and smooth floor cannot, therefore, be closely predicted.

(*ii*) On a sloping glacis, the position of the standing wave is definite and can be closely predicted.

(*iii*) The length of the jump is approximately five times the height. While the beginning of the jump is fairly definite, its lower end is indefinite. In estimating the length, the lower end is taken as the place where the water surface becomes and remains sensibly level. The position of this is variable and difficult to locate.

6.4.1. For a structure to be safe, the standing wave should be confined to the sloping glacis and not permitted to form on the horizontal beyond the toe of the glacis. The main disturbance of the standing wave dies out at distance from the point of formation equal to five times the jump or 5 (D_2-D_1) where D_1= depth of water just upstream of the point of formation of the wave and D_2=depth of water just downstrem of the point of formation of the wave. Therefore, in order that filter area and the stone protection be immune from the main turbulence of the standing wave the length of the horizontal floor should be nearly 5 (D_2-D_1).

6.4.2. The cistern level and its length have to be worked out for various sets of conditions imposed on the structure on the basis of the gate regulation proposed. The most severe condition would give the lowest cistern and its length. These are generally determined by the use of curves available for various discharge intensities and water depths or by analytical method. The levels and lengths of cistern determined by either of these methods have to be verified by 2*D*—hydraulic model tests for their efficacy. In energy dissipation calculations, retrogressed rating curve should be used as this gives deeper cistern levels.

6.4.3. *Determination of Cistern Level by the use of Curves.*

For determination of cistern levels, a set of curves known as Blench Curves (Figure 6.7) and Montague curves (Figure 6.8) extended by Kanwar Sain are generally used. The procedure is as follaws :

(*a*) From the proposed gate regulation for various discharges downstream of the river, the downstream water levels (Retrogressed) and the upstream water levels for passing those discharges plus canal withdrawals if any can be estimated. The total energy line values both upstream and downstream can be estimated by adding the head due to velocity of approach and exit.

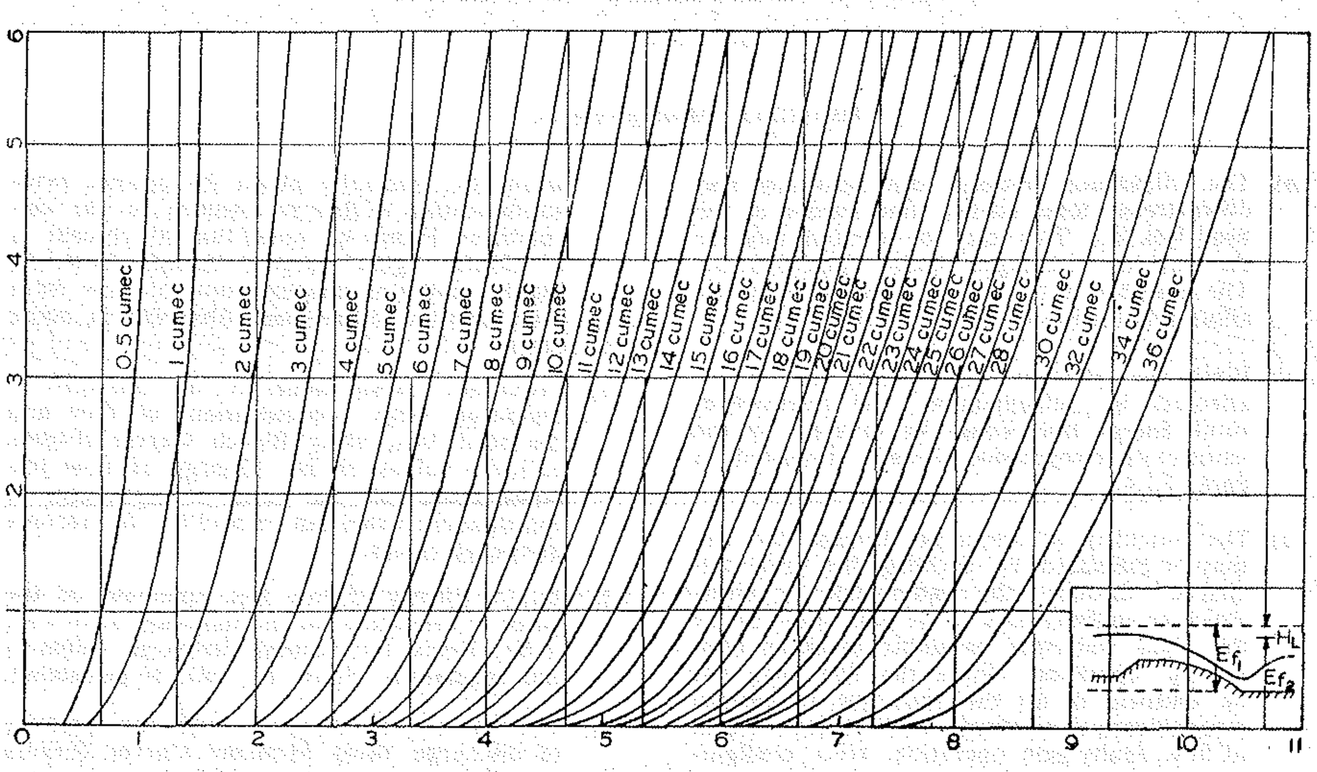


FIGURE 6.7 : Blench Curves.

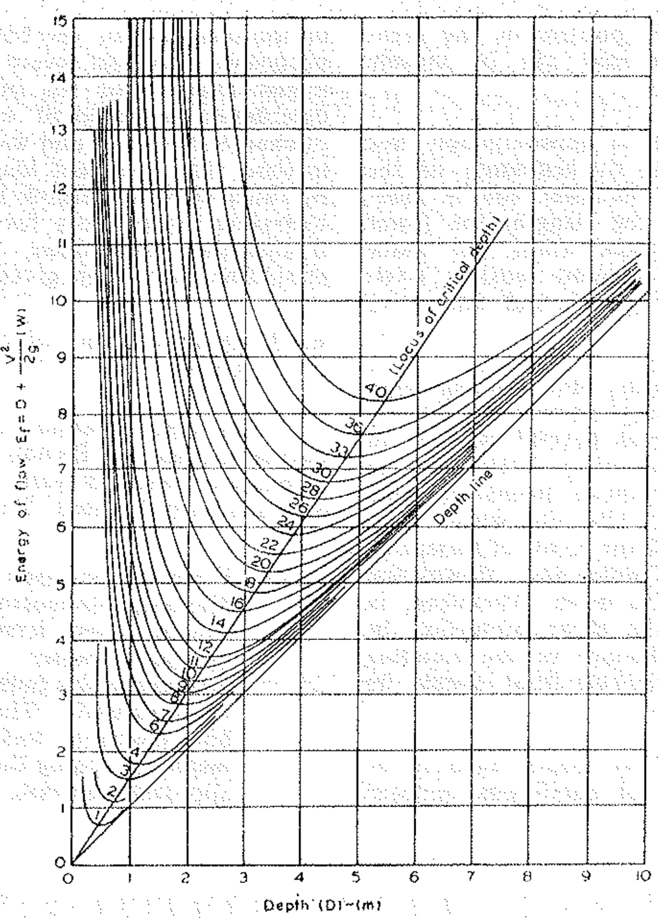


FIGURE 6.8 : Montague Curves.

(*b*) The difference between the upstream and downstream total energy line values is the head loss H_L. This may be worked out for the various discharges under consideration. The head loss at the design flood stage is the afflux provided in the diversion structure.

(*c*) While the upstream total energy line is unaffected by retrogression, the downstream total energy line would be lowered by the amount of retrogression already discussed in Para. 6.2.4.

(*d*) The intensity of discharge through the bay may be estimated by dividing the total discharge through the undersluice or riversluice or spillway bays as the case may be by the clear waterway available in those bays during operation. Since the flow may not be uniform in all the bays due to various reasons such as curvature of river flow, stages of flow, faulty gate operation, river configuration etc., generally about 20 percent extra concentration of flow is assumed in the calculations. It may be noted that 20 percent is a general figure adopted though, however, higher values of concentration of flow have been reported to have been observed in some cases.

(*e*) From the known values of the intensity of discharge with concentration of flow and the head loss, using Blench Curves (Figure 6.7), the values of E_{f_2} (Energy of flow just downstream of the point of formation of the standing wave) can be read off for various discharge values.

(*f*) E_{f_1}, i.e, Energy of flow just upstream of the point of formation of the standing wave $= E_{f_2} + H_L$. Hence, for various discharge values of the regulation chart, E_{f_1} can be calculated.

(*g*) For the known values E_{f_1} and the intensity of discharge, using Montagu Curves (Figure

6.8), the values of D_1 and D_2 i.e., the hyper-critical and sub-critical depths of water before and after the hydraulic jump can be read out.

(*h*) The required cistern level would be equal to the retrogressed water level for that discharge minus D_2. The required cistern length would be 5 (D_2-D_1).

6.4.4. *Determination of Cistern Level by Analytical Method*

The various steps involved in the determination of cistern level by analytical method are as follows :

(*a*) As mentioned in Para 6.4.3 (*a*), the total energy line values both upstream and downstream are estimated.

(*b*) An arbitrary cistern level for the particular discharge is initially assumed.

(*c*) E_{f_1}=Upstream T.E.L—Assumed cistern level.

(*d*) From the known values of E_{f_1} and q with concentration, D_1 can be calculated from the relationship,

$$E_{f_1}=D_1+\frac{q^2}{2gD_1^2}$$

where g is the accelaration due to gravity.

(*e*) Since the most important characteristic of the hydraulic jump is the pre-jump Froude's number F, it may be calculated from the relationship,

$$F^2=\frac{q^2}{gD_1^3}$$

(*f*) From the calculated values of D_1 and F, D_2 can be calculated from the relationship,

$$D_2=\frac{D_1}{2}\left(-1+\sqrt{8F^2+1}\right)$$

(*g*) The required cistern level would be equal to the retrogressed water level for that discharge minus D_2.

(*h*) It would be seen that the initially assumed cistern level (Step *b*) and the calculated cistern level (Step *g*) may not be the same which indicates that the cistern level to be assumed initially needs to be raised or lowered. After assuming a modified cistern level, the various steps should be repeated again till the assumed and calculated cistern levels tally with each other. The exact values of D_1 and D_2 will thus be known.

(*i*) The required cistern length would be 5 (D_2-D_1)

6.4.4.1. It would be observed that the cistern elevations determined by use of curves and by analytical method would vary. Hence, generally the analytical method is preferred as it gives more accurate results. Use of curves would be quicker and may be used for very preliminary computations.

6.4.5. *Prevention of Dangerous Scour Downstream of Aprons*

The length of the downstream horizontal floor can be reduced with a corresponding saving in cost, if the range of turbulence of the downstream wave can be restricted and the normal steady level of the downstream water obtained in a distance less than five times the jump from the toe of the glacis. The bed erosion on the downstrem side are caused by the occurrance of vertical and negative horizontal vortices. The former can be provided against by (*i*) constructing suitable lengths of divide walls between spillway and sluice bays and (*ii*) maintaining uniform intensity of discharge as far as possible. The negative horizontal vortices can be prevented by a number of devices which are designed to give an upward bend to the high velocity jets of the effluent water. These energy dissipating devices are generally in the form of

(*a*) a continuous end sill or Rehbock's dentated end sill.

(*b*) arrows.

(*c*) raised blocks

(*d*) a roughened or toothed floor

(*e*) a combination of two or more of these forms.

U.S.B.R. has standardised stilling basin designs for different values of Froude numbers. For details, any standard text books on Hydraulic Structures or publications on Energy Dissipation may be referred. One set of blocks called Chute blocks are also provided at the toe of downstream glacis.

While advantage can be taken of the provision of energy dissipating devices, generally these are provided extra to take care of any unknown flow pattern. In any case, however, the proposals for energy dissipating arrangements will need to be checked up by 2-D model studies for their efficacy.

Some of the energy dissipating devices usually adopted for diversion structures are indicated in Figures 6.9 and 6.10.

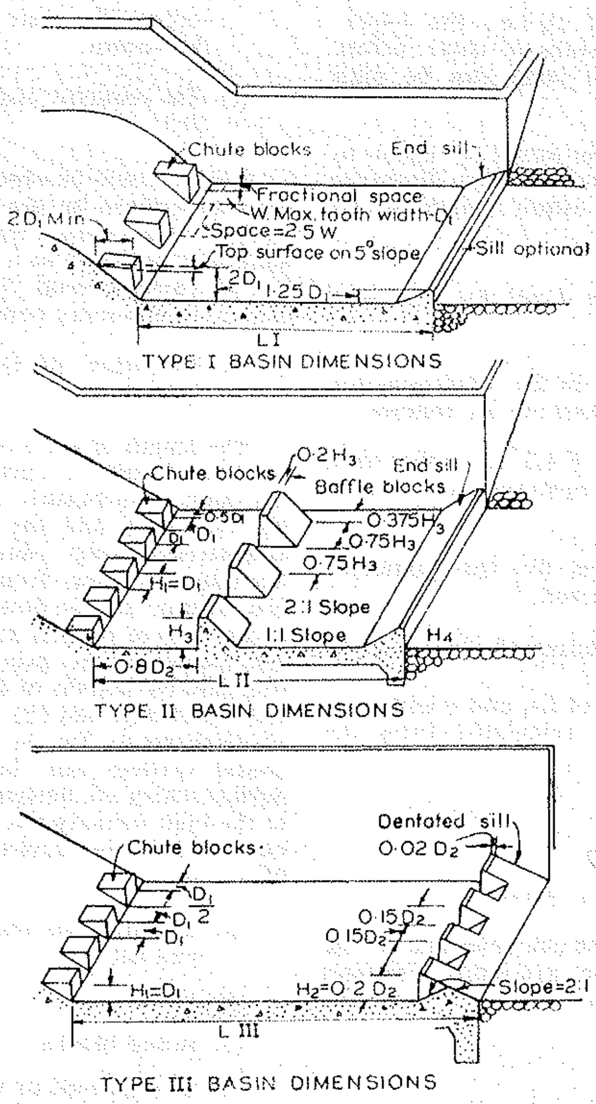


FIGURE 6.9 : Energy Dissipating Devices.

6.5. Upstream and Downstream Glacis

The glacis is the slope joining the crest with the floor on the upstream or downstream side as the case may be. From actual observations at site and from calculations, it has been found that flatter the glacis, the more intense is the wave and the greater the range of the trough, requiring heavy thickness along a much greater length of the floor. Theoretically, a stream requires more gradual expansion than contraction. From this, it follows that the downstream glacis should be flatter than that of the upstream. Where there is considerable heavy material rolling over the crest, a flatter upstream slope would be indicated. It has thus been observed that slopes between 1 : 3 and 1 : 5 for the glacis are considered to be the most suitable both for maximum dissipation of energy and economy.

6.6. Crest width

In the case of a broad crested weir, the crest width is usually governed by the requirements for traffic during fair weather if any and height of crest shuntters which when lowered are to be accommodated on the crest. The top width 't' may be taken from Bligh's empirical rule

$$t=\sqrt{H}+\sqrt{h}$$

where, h is the depth of water over the crest and H is the height of weir.

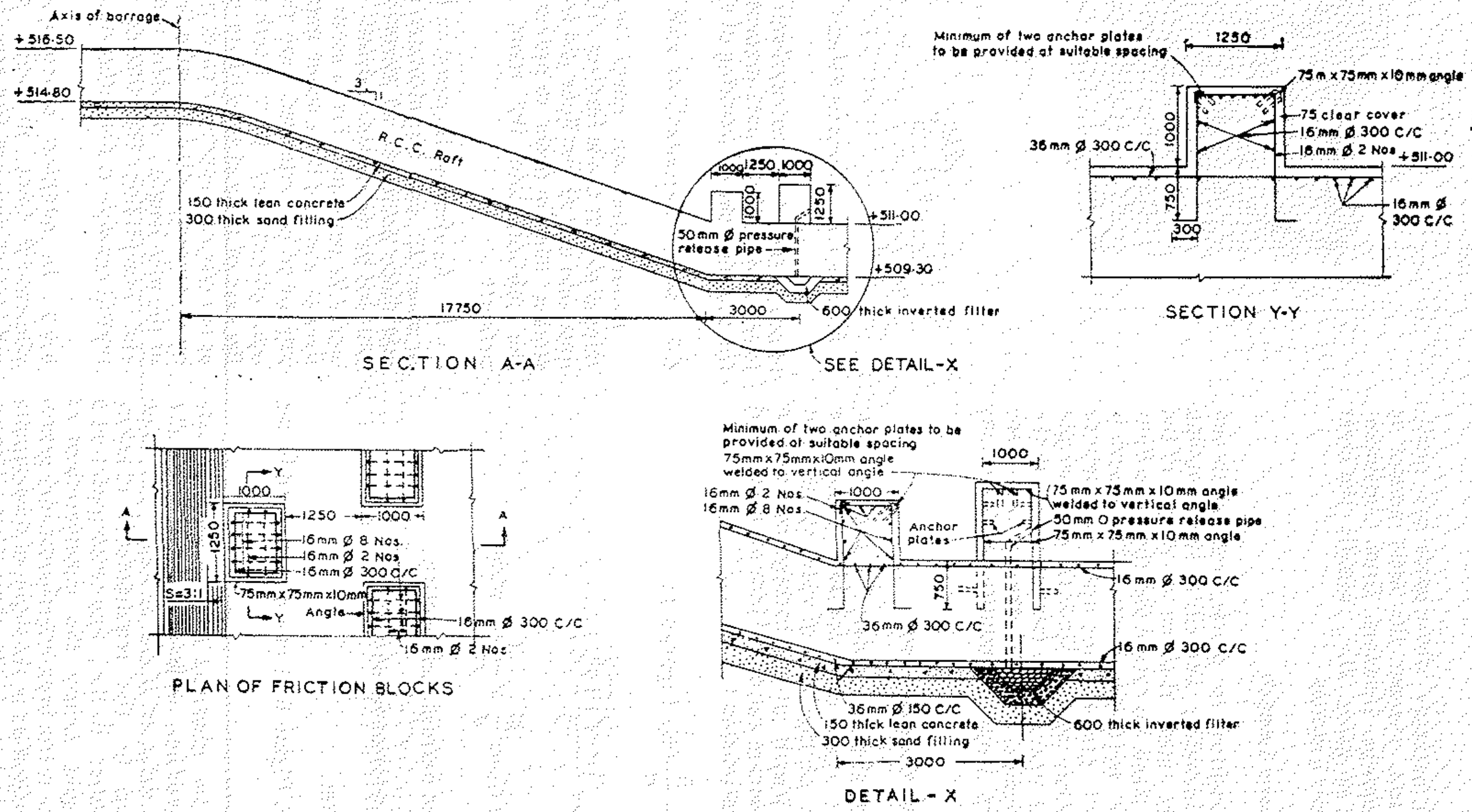


FIGURE 6.10 : Energy Dissipating Devices.

t has a minimum value of $3h/2w_1$ is the unit weight of masonry.

6.6.1. In the case of barrages where gates are provided instead of shutters, sharper the crest greater is the coefficient of discharge. However, from considerations of installation of gates, etc., usually the crest width is provided upto 2 metres.

6.7. Upstream Floor Length

From the discussions on glacis, crest width and energy dissipation including end sill in the foregoing paras, it would be seen that the top profile of the barrage/weir can be fixed up except the upstream floor beyond the toe of upstream glacis. This length is dependent on the exit gradient and uplift calculations explained in subsequent paras. Since the dead weight of water in the pond upstream of the gates adds to the stability of the diversion structure, the upstream floor length has to be fixed from the stability considerations also.

6.8. Exit Gradient Calculations

The exit gradient, already discussed in Para 6.1.5.4. is a very important aspect to be considered for a safe design. It depends on the head of water at the point of consideration, length of the barrage/weir section and depth of downstream cut off. These are connected by the following relationship :

$$\text{Ratio} = \frac{b}{d} \text{(See Figure 6.11)}$$

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

$$\text{Exit Gradient } G_E = \frac{H}{d} \times \frac{1}{\pi\sqrt{\lambda}} \text{ where}$$

b=length of the barrage/weir section

d=depth of downstream cut-off

H=head of water.

Curves given in Figure 6.11 can also be used in the calculation.

6.8.1. It would be observed that for getting a safe value of the exit gradient, different combinations of b and d for the same value of H can be had. The value of d is also governed by the requirements of safety against scour. Subject to this, any combination of b and d chosen should satisfy the considerations of economy also, since it has been experienced that generally for every 1m reduction in the depth of downstream cut off, b needs to be increased by about 10 m. This increase in the value of b has to be effected through out the

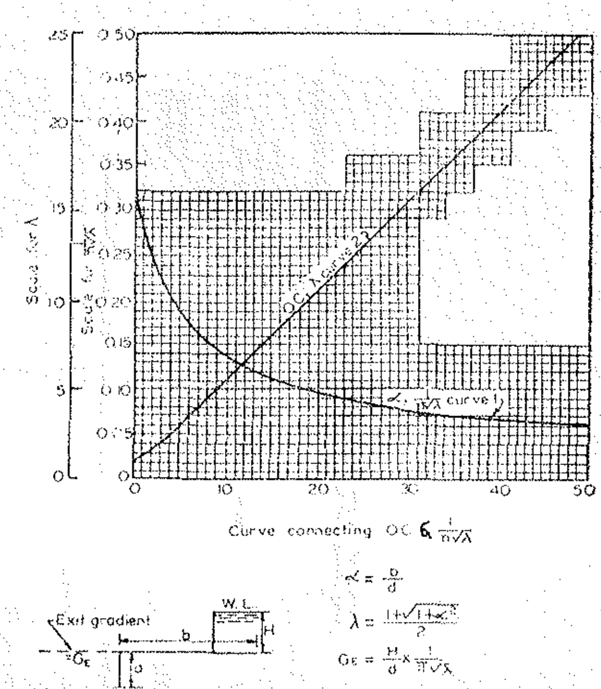


FIGURE 6.11 : Curves for Estimation of Exist Gradient.

entire width of the barrage/weir of in reaches where d is reduced with adequate precautionary measures taken for avoiding short-circuiting of seepage path.

6.9. Scour Calculations and Protection Works

Scour holes can occur both upstream and downstream so that a pile line/cut off line is required at the upstream end of the floor just as much as at the downstream end in order to prevent failure by slipping of the sub-soil into the scour holes by simple earth pressure.

6.9.1. As already discussed in Para 6.3.3.1., the value of scour 'R' can be calculated separately for spillway and sluice portions. 20 percent concentration factor will also be taken into account. The upstream cut off would be provided for a scour depth of R and downstream cut off for a scour depth of 1.25 R. The cut offs are to be extended into the banks on both sides upto atleast twice their depth from top of floors. In the case of alluvial soils where sheet piles are provided as cut offs, the depths of sheet piles would be increased by an amount called the grip length. For calculation of the grip length, standard books on Soil Mechanics and Foundation Engineering may be refered to. A typical example is also presented in Appendix 6.1.

6.9.2. In addition to the upstream and downstream pile/cut-off lines, in major structures, intermediate pile/cut-off lines are also provided. They are required neither to prevent undermining of the floor at the upstream nor at the downstream end, nor do they materially alter the pressure distribution to give less uplift pressures under the downstream floor. But they act as important secondary lines of defence so that even if the pucca floor is damaged at either end by failure of the end piles due to abnormal scour, the rest of the floor and the super-structure will be saved from collapase by the intermediate pile/cut off line. Generally two intermediate pile/cut off lines are provided, one between the upstream end and the crest and another between the downstream and the crest, located under the upstream and downstream toes of the glacis. While providing the intermediate pile/cut-off lines, it is to be ensured that the bottom level of the upstream intermediate lines is not higher than that of the upstream end line and that of the downstream intermediate line is not lower than that of the downstream end line.

6.9.3. In big barrages, the structures are constructed in a number of units/blocks, each comprising about 8 to 16 spans. It is necessary to ensure independent stability of each unit/block. For this purpose, cross pile/cut-off lines need to be provided in the longitudinal (flow) direction between the upstream and downstream lines. These are provided below the end piers of each unit/block which are part of the double piers. The bottom profile of the cross-pile/cut off line may follow the bottom profile of the pucca floor. Usually the bottom profile is in three portions (*i*) upstream portion having the same level as that of upstream cut off and extending from the upstream end to the toe of upstream glacis (*ii*) downstream portion having the same or slightly higher level as that of downstream cut off and extending from the downstream end to the toe of downstream glacis and (*iii*) transition portion joining the upstream and downstream portions. These are shown in Figure 6.12.

6.9.4. *Protections*

At the upstream end, a further protection of blocks over loose stone should be given so as to protect the soil adjoining the upstream end of the pucca floor. The length of this protection should be nearly equal D' which is the depth of scour hole below the floor. The bottom level of upstream scour hole is estimated by adopting a scour factor of 1.25 R to 1.75 R (mean 1.5 R). For calculation of R, reference may be made to Para 6.3.3.1. The thickness of this block protection over loose stones (also called spalls) should be the same as that of the stone apron at that end. Generally cement concrete blocks of 1.5 m × 1.5 m × 0.9 m size over 0.6 m thick loose stone are provided on the upstream side for the required length

At the downstream end, there should be an area of inverted filter of length equal to 1.5 D and depth equal to that of the downstream stone apron. This length is normally made up of deep blocks laid over graded filter of length 0.75 D and over loose stones of length 0.75 D. In between, a curtain wall of depth 0·5 m more than that of the stone apron is provided. Since wide shallow blocks are apt to be carried away by the current of water, deep blocks are to be employed which get wedged in and resist dislocation. The interslices (jharries) between the blocks are filled up with bajri For estimation of the value of D, a scour factor of 1.75 to 2.25 R (mean 2.0 R) may be adopted.

Beyond the block protections both on the upstream and downstream sides, loose stone aprons would be provided as discussed in Para 6.1.5.5. If D is the depth of scour hole below the apron level, the length of apron over a launching slope of 2 : 1 would be $\sqrt{5}D$. With a required thickness of 1 metre, the sectional area would be $\sqrt{5}D$. The apron as originally laid must have the same quantity of stone and hence the sectional area. The length of stone protection may vary from 1.5 D to 2.5 D (higher figure for flatter slopes of the river). Accordingly the thickness of the stone apron may be calculated. Usually it is about 1.5 metres. The scour factors adopted for the upstream and downstream block protections hold good for the loose stone apron also. The appron is provided in the form of a wedge, with smaller thickness inside.

6.9.4.1. It may be noted that while computing the value of R no concentration (20%) need be taken in the value of q in the case of upstream and downstream protection works.

6.9.4.2. Typical calculations for scour and protection works are given in Appendix 6.1.

6.10. Uplift Pressure Calculations

After fixing the profile of the diversion structure including the levels of the pile/cut off lines, the thickness of the pucca floor at various places like upstream floor, crest, glacis, downstream stilling basin, etc,. may be suitably assumed and the percentage of seepage pressure below the floor estimated at salient points. Necessary corrections on account of the thickness, mutul interference of piles and slope of glacis will need to be applied wherever necessary. After this, the required thickness of floor to be safe against uplift pressure has to be calculated at the salient points, as explained in the relevant para of Chapter 7 on Structural Designs. Under Hydraulic Designs, only the procedure for estimation of the percentage pressures is outlined. From the thickness calculated under structural Designs, the assumed thickness of the floor at various points may be suitably modified and for this final proposed profile of the floor, the percentage pressures may be estimated again.

6.10.1. The percentage pressures at the sheet pile points are estimated from the curves given in Figure 6 3. In this Figure (*a*) will be used for estimation of percentages at piles at the upstream and downstream

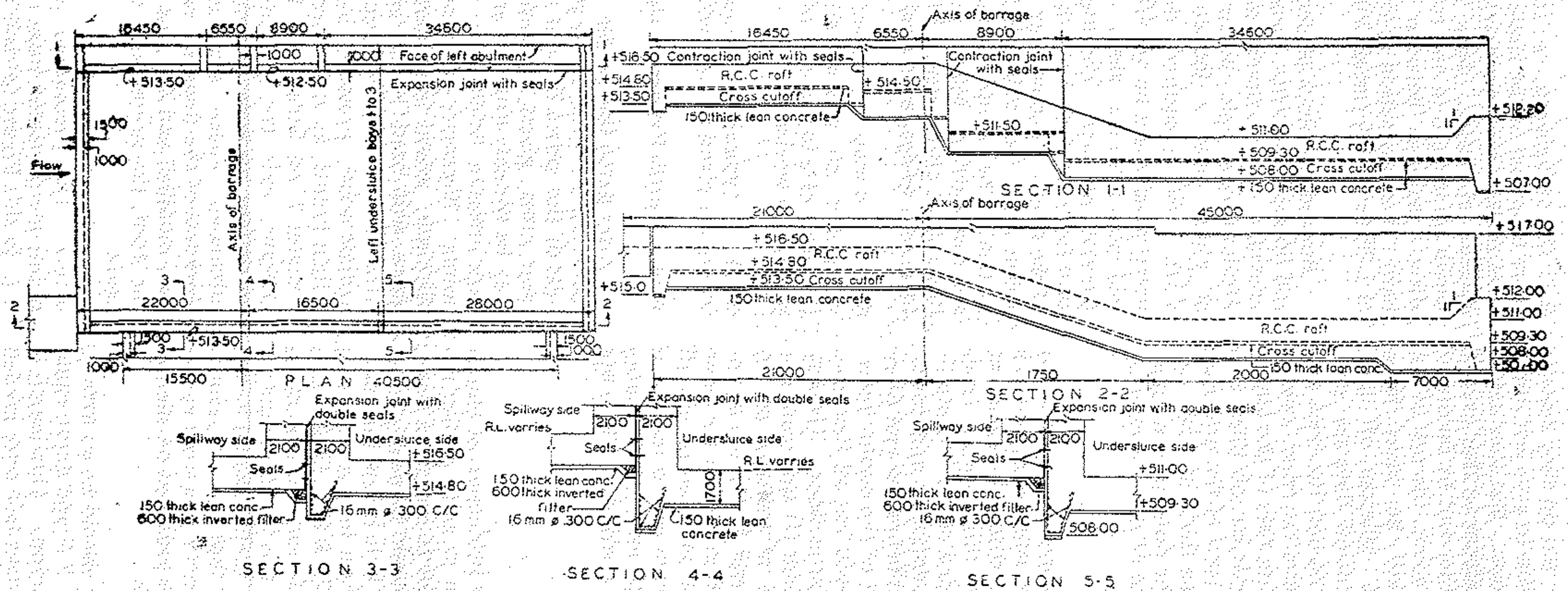


FIGURE 6.12 : Typical details of cross cut-off.

ends, (*b*) for the percentages at intermediate piles and (*c*) for slop corrections. The procedure is outlined below :

(*a*) *Sheet Piles at Ends.* Curves for ϕ_D and ϕ_E *Vs* $\frac{1}{\alpha}$ have been presented. $\frac{1}{\alpha}$ is the ratio $\frac{d}{b}$ = depth of pile below floor level/length of the section of spillway bay under consideration. For the downstream end, the depth of pile is measured from the cistern level. For up tream end, the percentage pressure ϕ_{D_1} at the bottom of the pile and ϕ_{C_1} at the top of the pile on the inner side would be 100 minus the values of ϕ_D and ϕ_E read off from the curves for the upstream at value of $\frac{1}{\alpha}$. For the downstream end, the percentage pressure ϕ_{D_2} at the bottom of the pile and ϕ_{E_2} at the top of the pile on the inner side would be the values of ϕ_D and ϕ_E read off from the curves for the downstream values of $\frac{1}{\alpha}$.

The values thus estimated would be corrected for thickness definitely and for interference of intermediate piles if applicable. As the percentage pressure is required to be estimated at the bottom of the floor and not at the top as read off from the curves, the correction for thickness is applied, assuming the variation between the bottom and top of the pile to be linear. Thus the correction for thickness would be equal to $\frac{\text{Thickness of floor}}{\text{Depth of pile}} \times$ Difference in percentages at top and bottom.

The correction to be applied to the value at the top of floor would be positive in the case of upstream pile and negative in the case of downstream pile.

Wherever applicable, correction for mutual interference would be applied as explained in Para 6.1.5.3 (*b*).

(*b*) *Intermediate Sheet Pile.* Curves for ϕ_D and ϕ_E *Vs.* $\frac{bi}{b}$ have been presented for different values α. α is the ratio $\frac{b}{d}$ = length of the section/depth of intermediate pile from the upstream floor level or downstream cistern level as the case may be. b_1 is the distance of the pile from the upstream end of the floor. While reading off the values from the curves, the following point are to be noted :

(*i*) To find ϕ_E for any value of α and base ratio $\frac{bi}{b}$, ϕ_C may be read for base ratio $\left(1-\frac{b_1}{b}\right)$ for that value of α and subtracted from 100.

(*ii*) To find ϕ_D for values of $\frac{b_1}{b}$ less than 0·5, the value of ϕ_D for the base ratio $\left(1-\frac{b_1}{b}\right)$ may be read and subracted from 100.

Generally the values so estimated for the intermediate piles will have to be corrected for thickness, mutual interference of piles and slope. The slope corrections are to be applied as explained in Para 6.1.5.4. (*c*).

6.10.2. The percentage pressures at the bottom of the floor between the piles in various reaches are assumed to vary linearly and accordingly the percentages at salient points can be estimated for use in the Structural Designs.

6.10.3. The percentage pressures estimated on the guidelines outlined in the forgoing paras pertain to two dimensional flow and for a homogeneous bed of indefinite thickness. However, actual field conditions may be different with finite thicknesses of permeable foundations and also three demensional flow. These can be termed as special seepage problems and have been dealt with in Chapter 20.

6.11. Divide Walls

The functions and general layout of divide walls have been briefly discussed in Para 5.2.6 and its subparas While the final lengths and alignments of the divide walls are to determined by hydraulic model tests, the following guidelines would help in the initial fixation of their lengths :

(*i*) Experiments conducted at the Centre Water and Power Research Station, Pune, indicate that a divide wall should not extend beyond the upstream end of the head regulator and generally satisfactory results are obtained if the divide walls covers two-thirds the width of the head regulator. Similar results have been obtained by the Irrigation Researeh Institute, Roorkee also.

(*ii*) The length of the downstream divide wall should be such as to prevent the development of any parallel flow along the structure under various flood stages Usually the downstream divide will is extended up to the end of the downstream apron. Sometimes, model experiments indicate their omission also.

(*iii*) The upstream divided wall may also be kept for such a length that the heavy attack on its nose may be kept away from the upstream protection of the sluice bays.

(*iv*) The curvature of the river at the site of head-works play an important part in determining the length of the divide wall. Thus, if the pocket is at the outer curvature, the length of the divide walls could be reduced with advantage. Otherwise, they may do more harm than good.

6.12. Typical hydraulic calculations covering the aspects on waterway, energy dissipation, exist gradient, scour and protections and uplift pressure percentages for the main structure are presented in Appendix 6.1.

APPENDIX 6.1

Typical Hydraulic Calculations

Maximum Flood Discharge

$=3146$ cumecs

Lacey's regime width

$=4{\cdot}83\sqrt{Q}=4{\cdot}83\sqrt{3146}$

$=271$ m.

Total width provided between abutments

=(Under sluice $3\times6{\cdot}1$)+(River sluice $4\times6{\cdot}1$)+7 piers of 1·52 m each+Fish Ladder 2·13 m+ Fish Ladder Pier 1·52 +Spillway 241·8 m

=298·65 m

Looseness Factor=298·65/271=1·1

The velocity of approach is assumed to be constant over spillway, river sluices and under sluices.

Intensity of discharge

$$q=\frac{3146}{298{\cdot}65}=10{\cdot}53 \text{ cumecs/m}$$

Scour depth $R=0{\cdot}475\left(\frac{Q}{f}\right)^{1/3}=6{\cdot}96$ m

$f=1$

Velocity of approach

$$V_a=\frac{10{\cdot}53}{6{\cdot}96}=1{\cdot}51 \text{ m/sec.}$$

Head due to $V_a=0{\cdot}177$ m,

D/S HFL from G-D curve

=94·21 m

Assume Afflux as 0·6 m

During high floods, according to the lay out envisaged, the under sluices function like drowned orifice and river sluices and spillway function like submerged weir.

Crest level of undersluice =89·48

Crest level of River sluice =89·48

Crest level of Spillway =91·62

R.L. of bottom of Breast-well in undersluices =92·84

Discharge through undersluices

U/S HFL=94·21+0·6=94·81 m

D/S HFL=94·21 m

Using Drowned orifice formula

$$Q=C_d.\ \text{B.H.}\ \{\sqrt{2g\,(H_1-H_2)+V_a}\}$$

where $B=3\times6{\cdot}1=18{\cdot}3$ m

$H=92{\cdot}84-89{\cdot}48+3{\cdot}36$ m

$H_1-H_2=0{\cdot}6$ m

$V_a=1{\cdot}51$ m/sec.

$Q_{u.s}=197$ cumecs

Discharge through River Sluices

$$\text{Drowning Ratio}=\frac{94{\cdot}21-89{\cdot}48}{94{\cdot}81-89{\cdot}48}=\frac{5{\cdot}73}{6{\cdot}33}=0{\cdot}9$$

Coefficient of discharge

$C=2{\cdot}76$ (from curves—British units)

Using the equation

$$Q=\text{C L.}\ \{(H+H_a)^{3/2}-H_a^{3/2}\}$$

where $L=4\times6{\cdot}1=24{\cdot}4$ m

$H_a=0{\cdot}177$ m

$H=6{\cdot}33$ m

$Q_{river\ sluice}=482$ cumecs

Discharge through spillway

$$\text{Drowning Ratio}=\frac{94{\cdot}21-91{\cdot}62}{94{\cdot}81-91{\cdot}62}=\frac{2{\cdot}59}{3{\cdot}19}$$

$=0{\cdot}81$

Coefficient of dischare

$C=2{\cdot}96$ (from curves—British units)

Using the equation. $Q=\text{C.L.}\{(H+H_a)^{3/2}-H_a^{3/2}\}$

where $L=241{\cdot}8$ m

$H_a = 0{\cdot}177$ m

$H = 3{\cdot}19$ m

$Q_{spillway} = 2452$ cumecs

Total discharge through undersluices+River-sluices+Spillway $= 197+482+2452$

$= 3131$ cumes ≈ 3146 cumecs

D/S Retrogressed WL $= 94.21 - 0{\cdot}46$

$= 93{\cdot}75$ m

Assume Cistern level as 89·94 m

$E_{f_1} = 94{\cdot}81 - 89{\cdot}94$

$= 4{\cdot}87$ m

$$= d_1 + \left(\frac{12{\cdot}17}{4.43\, d_1}\right)^2$$

Discharge Total in cumecs	Canal Dis. cumecs	Undersluice Dis. cumecs	Riversluices Dis. cumecs	Spillway Dis, cumecs	U.S. W.L.
0—210	18·4	Operated occasionally to scour bed	191·3	0	93
210—567	18·4—0	,,	191·3—51	0—516	93·66
517—1134	0	0—2·95	51—243	516—947	93·78
1134—1700	C	29·5—119	243—312	947—1355	94·05
1700—2267	0	119—151	312—374	1355—1768	94·36
2267—3117	0	151—197	374—482	1768—2451	94·81

Exact Afflux value can be worked out so that the total discharge is near about 3146 cumecs.

Using similar procedure. the discharges through the undersluices, siver sluices and spillway for various total discharges of the river can be estimated. These values can be used for cistern level calculations. In this particular example where falling shutters are provided over spillway crest, the following values are obtained.

Cistern level calculations

Total River discharge $= 3117$ cumecs

Max. discharge thro' spillway$= 2451$ cumes

Discharge intensity $q = \dfrac{2451}{241{\cdot}8}$

$= 10{\cdot}14$ cumec/m

q with 20% concentration $= 12{\cdot}17$ cumec/m

U/S W.L. $= 94{\cdot}81$ m

$\therefore \quad d_1 = 1{\cdot}48$ m

$$F^2 = \frac{12{\cdot}17^2}{9{\cdot}8 \times 1{\cdot}48^3} = 4.66$$

$$d^2 = \frac{1{\cdot}48}{2}\left\{-1 + \sqrt{8 \times 4{\cdot}66 + 1}\right\}$$

$= 3{\cdot}83$ m

Reqd. cistern level$= 93{\cdot}75 - 3{\cdot}83$

$= 89{\cdot}92$ m —OK

Cistern length reqd.

$= 5(d_2 - d_1)$

$= 5(3{\cdot}83 - 1{\cdot}48)$

$= 11{\cdot}75$ m.

Similar calculations for various discharge values through spilling, River sluices and undersluices can be worked out. For each component the governing values of cistern level and cistern length will be determined after tabulating the calculations.

Sheet Pile Level Calculations

(Also the Appendix 7·1)

For Spilllway, max. discharge

$=2451$ cumecs

Intensity of discharchge

$$q=\frac{2451}{241\cdot8}=10\cdot14 \text{ cumecs/m}$$

q with concentration of 20%

$=12\cdot17$ cumecs/m

$$R=0\cdot475\left(\frac{Q}{f}\right)^{1/3}$$

$$=0\cdot475\left(\frac{1\cdot2\times2451}{1}\right)^{1/3}$$

as looseness factor $B>1$

$=6\cdot81$ m.

Bottom of U/S steel pile

$94\cdot81-6\cdot81$

$=88\cdot00$

(See Appendix 7·1 per grip length, etc.)

Taking the scour factor as 1·25 R for the downstream.

$1\cdot25\ R=1\cdot25\times6\cdot81=8\cdot51$ m

Bottom of D/S sheet pile

$=93\cdot75-8\cdot51=85\cdot24$

However this is to be checked for safe exit gradient also.

Similar calculations can be carried out for other components also.

Exit Gradient Calculations

For Spillway

Checking for pond level condition and checking at top of blocks,

$H=93-88\cdot42$

$=4\cdot58$ m $=88\cdot42$ is Min WL.

depth of d/s sheet pile

$=90\ 55-84\cdot76$

$=5\cdot79$ m

$b=27\cdot9$ m

$$\alpha=\frac{b}{d}=\frac{27\cdot9}{5\cdot79}=4\cdot82$$

From curves, $\dfrac{1}{\pi\sqrt{\lambda}}=0.19$

Exit Gradient $=\dfrac{4.58}{5.79}\times0.19=0.15=\dfrac{1}{6.7}$ safe

At bottom of filter, calculating in the same way

Exit Gradient $=\dfrac{1}{5\cdot7}$ safe.

Similar calculations for exit gradient are to be done for under sluice and river sluice also.

C.C. Block and Loose Stone Protection

Spilllway—Max. intensity of discharge

$=10\cdot14$ cumecs/m

With concentration $q=12\cdot17$ cumecs/m

[Note. In case of spilllway with gate and piers, total width of spillway is to be taken for computing q and no concentration to be taken.]

Scour depth $R=6\cdot81$ m

$1\cdot5R=10\cdot22$ m

Bottom of Scour hole

$=94\cdot81-10\cdot22=\cdot85\ 59$ m

Depth of scour below floor

$D=91\cdot62-83\cdot59=8\cdot03$ m on U/S.

Provide 4 rows of $1\cdot5$ m $\times$ $1\cdot5$ m $\times$ $\cdot9$ m C.C. blocks on U/S

$2R=13\cdot62$ m

Bottom of Socur hole

$=93\cdot75-13\cdot62=80\cdot13$ m

Depth of scour below floor

$D=90\cdot55-80\cdot13$

$=10\cdot42$ m

$1\cdot5\ D=15\cdot63$ m

Provide 5 rows of $1\ 5$ m $\times$ $1\cdot5$ m $\times$ $\cdot9$ m C.C. blocks over inverted filter $+1\cdot5$ m wide curtain wall $+5$ rows of C.C. blocks over stone spawls.

Launching slope $=2:1$ assumed.

Slope length $=D\sqrt{5}=8\cdot03\sqrt{2}$ m on U/S.

$=18$ m.

Taking 1·2 m thick loose stone from slope considerations.

Quantity of stone required

$$=18\times1{\cdot}2=21{\cdot}6\ m^3$$

With 1·5 m thick apron length

$$=\frac{21{\cdot}6}{1{\cdot}5}=14{\cdot}4\ m$$

$1{\cdot}5\ D$ on U/S. $=8{\cdot}03\times1{\cdot}5$ m

$$=12\ m$$

Provided length $=15$ m on U/s.

Similarly on d/s side, quantity of stone reqd $=28\ m^3$

With 1·5 m thick apron, length

$$=\frac{28}{1{\cdot}5}=18{\cdot}67\ m$$

Provided length $=20$ m

Similar calculations are to be done for under-sluices, riversluices, around divide walls (shanks and noses) and guide bunds (shaks and noves).

Uplift Pressure Calculation

Taking a typical case with u/s, d/s and two intermediate rows of sheet pile, the uplift pressure calculations are presented here.

First row of sheet pile (u/s row)

R.L. of bottom of sheet pile

$$=-1{\cdot}22\ m$$

$$b_1=0{\cdot}61\quad b=73{\cdot}17$$

$$d=\text{Floor level }13{\cdot}11-(-1{\cdot}22)$$

$$=14{\cdot}33\ m$$

$$\alpha=\frac{73{\cdot}17}{14{\cdot}33}=5{\cdot}1$$

$$b_1/b=0{\cdot}61/73{\cdot}17=0\ 00834$$

$$1-b_1/b=1-0{\cdot}00834=0.99166$$

$$\varphi_C=61.8\%$$

$$\varphi_E=100-1=99\%$$

$$\varphi_D=100-26.2=73.8\%$$

Correction for thickness of floor

For $\qquad \varphi_C=\dfrac{73.8-61.8}{14.33}\times2.134=1.79\%$

For $\qquad \varphi_E=\dfrac{99-73.8}{14\ 33}\times2.134=3.84\%$

Correction for interference of 2nd row of pile line.

$$=\frac{42\ D}{b}$$

$$=42\times\frac{10.67}{42.68}=10.5\%$$

Corrected $\quad \varphi_C=61.8+1.79+10.5=74.09\%$

Corrected $\quad \varphi_E=99-3.84\qquad =95.16\%$

Second row of sheet pile (1st intermediate row)

Bottom level of 2nd row of sheet pile

$$=+0.305\qquad(\text{Depth}=10.67\ m)$$

$$b_1=17.07\ m$$

$$b=73.17\ m$$

$$d=13.11-0.305$$

$$=12.805\ m$$

$$\alpha=b/d=73.17/12.805=5.725$$

$$b_1/b=17.07/73.17=0.233$$

$$1-b_1/b=1-0.233=0.767$$

$$\varphi_C=54\%$$

$$\varphi_E=100-21=79\%$$

$$\varphi_D=100-35=65\%$$

Correction for floor thickness

For $\qquad \varphi_C=\dfrac{65-54}{12.805}\times2{\cdot}134=1.83\%$

For $\qquad \varphi_E=\dfrac{79-65}{12.805}\times2.134=2.34\%$

Correction for interference of 1st row of sheet pile on φ_E.

$$=\frac{42D}{b_3'}=\frac{42}{42.68}\times[10.98-(-1.22)$$

$$=12\%$$

Correction for interference of 3rd row of sheet pile on φ_C.

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

$$D=10.98-(-4.57)=15.55$$

$$d=10.98-.305=10.675$$

$$=19\sqrt{\frac{15.55}{26.22}}\times\frac{10.675+15.55}{73.17}$$

$b_1 = 26.22$ m

$b = 73.17$ m

$= 5.24\%$

Slope correction for 3 : 1 slope

$= 4.5\%$

Length of the slope

$= 4.57$ m

Spacing of sheet piles

$= 26.22$ m

% correction $= \frac{4.5 \times 4.57}{26.22} = 0.785\%$

$\therefore$ Corrected $\varphi_C = 54 + 1.83 + 5.24 - .785$

$= 60.285\%$

Corrected $\varphi_E = 79 - 2.34 - 12$

$= 64{\cdot}66\%$

Third row of sheet pile (2nd intermediate row)

Bottom level of 3rd row of sheet pile

$= -4.57$ m

$b_1 = 43.29$ m

$b = 73.17$ m

$d = 8.23 - (-4.57) = 12.8$ m

$\alpha = b/d = 73.17/12.8 = 5.725$

$b_1/b = 43.29/73.17 = 0.592$

$1 - b_1/b = 1 - 0.592 = 0.408$

$\varphi_C = 33\%$

$\varphi_E = 100 - 44.5 = 55.5\%$

$\varphi_D = 44.5\%$

Correction due to floor thickness

For $\varphi_E = \frac{55.5 - 44.5}{12.8} \times 2.134 = 1.83\%$

For $\varphi_C = \frac{55.5 - 33}{12.8} \times 2.134 \times 3.75\%$

Correction due to interference of 2nd row of sheet pile on φ_E.

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

$D = 6.1 - .305 = 5.795$ m

$d = 6.1 - (-4.57) = 10.67$ m

$$= 19 \sqrt{\frac{5.795}{26.22}} \times \frac{10.67 \times 5.795}{73.15}$$

$b_1 = 26.22$ m

$b = 73.17$ m

$= 2.01\%$

Correction due to interference of 4th row of sheet pile on φ_C.

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

$D = 6.1 - (-4.57) = 10.67$ m

$d = 6.1 - (-4.57) = 10.67$ m

$$= 19 \sqrt{\frac{10.67}{29.27}} \times \frac{10.67 + 10.67}{73.17}$$

$b_1 = 29.27$ m

$b = 73.17$ m

$= 3.34\%$

Correction due to slope

$= 4.45$ for φ_E

$= 4.45 \times \frac{17.42}{26.22} = 2.96\%$

Corrected $\varphi_C = 33 + 3.75 + 3.34 = 40.1\%$

Corrected $\varphi_E = 55.5 - 1.93 - 2.01 + 2.96$

$= 54.62\%$

Fourth row of sheet pile (d/s row)

Bottom level of 4th row of sheet pile

$= -4.57$ m

$b_1 = 72.56$ m

$b = 73.17$ m

$d = 8.23 - (-4.57) = 12.8$ m

$\alpha = b/d = 73.17/12.8 = 5.715$

$b_1/b = 72.56/73.17 \cdot 0.99$

$1 - b_1/b = 1 - 0.99 = 0.01$

$\varphi_C = 1\%$

$\varphi_E = 100 - 63 = 37\%$

$\varphi_D = 25.5\%$

Correction for thickness of floor

For $\varphi_C = \dfrac{25.5-1}{12.8} \times 2.134 = 4.98\%$

For $\varphi_E = \dfrac{37-25.5}{12{\cdot}8} \times 2.134 = 1.92\%$

Correction for interference of 3rd row of sheet pile on φ_E.

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

$D = 6.1-(-4.57) = 10.67$ m

$d = 6.1-(-4.57) = 10.67$ m

$$= 19\sqrt{\frac{10.67}{29.27}} \times \frac{10.67+10.67}{73.17}$$

$b_1 = 29.27$ m

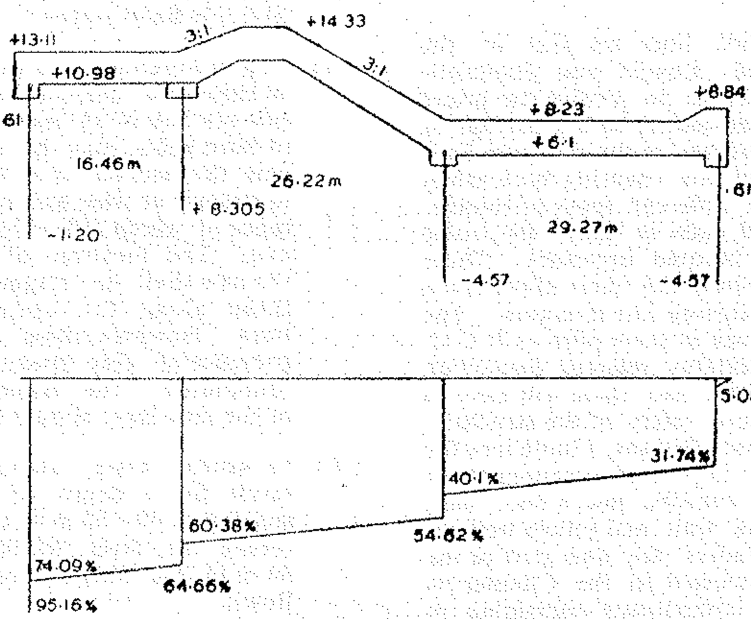
$b = 73.17$ m

$= 3.34\%$

Corrected $\varphi_C = 1+4.08 = 5.08\%$

Corrected $\varphi_E = 37.0-1.92-3.34$

$= 31.74\%$



UPLIFT PRESSURE PENCENT DIAGRAM

FIGURE 6.13 : Uplift Pressure % Diagram.

CHAPTER 7

Structural Designs

7.0 Introduction

From the various guide lines set out in the Chapters on Planning and Layout and Hydraulic Designs, it would be possible to evolve the broad features of the diversion structure, be it a barrage or a weir. The various components of the structure have to be very carefully designed for ensuring both safety and economy. Due to the different types of foundation materials encountered, such as clay, etc., some foundation treatments may become necessary. These will have to be properly evolved and their effect taken into consideration while designing the structure. The type of cuts-offs proposed such as sheet piles or R.C.C. Cut-offs will have to be suitably selected depending on the construction difficulties and these will have to be carefully designed to ensure safety of the structure. In this Chapter on Structural Designs, Guidelines for the design of some of the important components of the main structure such as cut-offs, pucca floor, pier, divide wall, abutment, flank wall and return well are set out. Though silt excluders are also part of the main structure, these are discussed in the Chapter on Silt Exclusion. The other components pertaining to the head regulator, fish pass, navigation facilities and river training works are discussed under relevant chapters.

7.1 Data Required

In addition to the data listed in Chapter 3 and analysed values thereof, the following data are also required to be collected for carrying out the Structural Designs. There are some repetitions. However, these have been included for convenience.

(*a*) High Flood level and Minimum water level in the river.

(*b*) Design Pond Level

(*c*) The lowest and highest tide levels in case of tidal streams.

(*d*) Intensity of silt charge in the river during high and low flood stages.

(*e*) Log of boreholes to a depth of about 15 to 25 m below the deepest bed level at a spacing of one per bay in at least three rows. One row of bore holes may be along the barrage/weir axis, the second at a distance of about 15 m upstream of the axis and the third at a distance of about 30—40 m downstream of the axis. The location of the boreholes along the axis shall be staggered with reference to those along the upstream and downstream lines. However, these are essential wherever presence of clay stratum is detected in the foundation. The extent, depth and location of the clay layer should be correctly assessed.

(*f*) In sandy strata, standard penetration test result for a depth of atleast 8 to 12 m at a spacing of 40—50 m in the transverse direction (along the river width) and at a spacing of 30 m in the longitudinal direction (along the flow).

(*g*) Wherever clayey strata are encountered, undisturbed samples of the clay layers from the proposed foundation level upto 8 metres below the foundation level for each bay. The analysed values of shear parameters, void ratio, consolidation characteristics, moisture content, in-situ density, sensitivity and permeability.

(*h*) For sandy soil, grain size distribution curves from undisturbed samples obtained at 3 metre intervals from each bore hole. Dry densities, relative densities and angle of internal friction.

(*i*) Modulus of subgrade reaction at the proposed foundation level. If the structure is long or if there is wide variation in the properties of the foundation materials, the value of modulus of sub-grade reaction in each unit/block separated by double pier. If it is not fessible to determine the modulus of subgrade reaction at site, the values given in Appendix 7.3 may be adopted for design purposes, depending on the type of soil met with.

(j) Results of feasibility studies for driving sheet piles or otherwise if the river bed consists of boulders or is made up of stiff soil.

7.2. Design of Cut-offs

The upstream and downstream cut-offs of the diversion structure may be of steel sheet piles anchored to the pucca floor by means of R.C.C caps or of masonry or R.C.C. The depth of cut-offs are fixed as discussed in Para 6.9.1.

7.2.1. The sheet pile cut-offs would be designed as sheet pile retaining walls anchored at top and to resist the worst combination of forces and moments, considering the possible scour on the outer side and earth pressure and surcharge due to floor loads on the inner side, differential hydrostatic pressure computed on the basis of the percentage of pressure of seepage flow below the floor, etc. If the effect of the cut-offs are taken into account for resistance against forward sliding of the structure, they have to be designed to withstand the passive pressures developed. The R.C.C. pile caps would be designed to transmit the forces and moments acting on the steel sheet pile cut-offs to the pucca floor.

7.2.2. The masonry or R.C.C. cut offs also would be designed to resist the forces and moments specified for sheet pile cut-offs and they would be treated as Cantilever walls fixed monolithically with the pucca floor.

7.2.3. The upstream intermediate cut-off usually has the same section as the upstream end cut-off and the downstream intermediate cut-off same as the downstream end cut-off.

7.2.4. The required sections of the sheet pile cut-off would be worked out and the nearest section subject to practical availability would be used for construction.

7.2.5. Typical example of design of sheet pile cut-off is presented in Appendix 7.1.

7.3. Design of Impervious (Pucca) Floor

The impervious or pucca floor of the diversion structure may be of two types, namely, gravity type and reinforced cement concrete raft type. In the gravity type, the uplift pressure is balanced by the self-weight of the floor only considering unit length of the floor. In the raft type, the uplift pressure is balanced by the weight of the floor, piers and other super-imposed dead loads considering the portion of the raft between joints as single unit. The gravity floor can be of plain concrete or masonry. The top profile of the floor can be fixed up based on the guide lines set out under Hydraulic Designs. While the uplift pressure percentage under the floor can be estimated as per Para 6.10 and its sub-paras, the profile of the hydraulic jump under High Flood and Pond Level conditions can be worked out from Montague Curves (Figure. 6 8) by finding the values of the depth of the flow over the profile of the floor.

7.3.1. *Gravity Floor*

The adequacy of the thickness of the floor has to be checked for the three conditions, viz., (*i*) No flow downstream which gives the maximum static head (*ii*) high flood and (*iii*) flow at pond level. Under (*ii*) and (*iii*) hydraulic jump may form over the downstream glacis. From the percentage pressures estimated already, the levels of the hydraulic gradient lines at key points under different flow conditions can be worked out. Both the hydraulic gradient lines and jump profiles are to be plotted neatly.

7.3.1.1. The floor thickness at any point should not be less than the maximum ordinate between the sub-soil hydraulic gradient line and the water surface (or floor surface if there is no flow) divided by $(G-1)$ where G is the specific gravity of the material of which the floor is made of such as masonry or concrete. The specific gravity of concrete is taken as 2.25. On the glacis, the maximum of the ordinates under any of the three conditions would govern the floor thickness at that key point. The thickness of the floor adopted for construction should be at least 10% more than the thickness required calculated as above.

7.3.1.2. The uplift pressure in the jump cistern and the requirement of floor thickness are moderated to some extent by the following factors :

(*i*) Due to a backward rolling of flow of water into the jump cistern, it is partially filled up reducing the unbalanced uplift pressure.

(*ii*) The uplift due to maximum depth of the cistern would operate only at the deepest point of the trough, becoming smaller on either side. As the floor has always got some beam action, it may be designed in small lengths (of about 1 to 2 m) for average uplift on each strip, rather than for the maximum uplift at every point. As the jump is likely to travel over practically the entire glacis for varying discharges this allowance for spread results in considerable economy in the floor thickness.

(*iii*) The vertical part of momentum remains unaffected at the jump and results in an impact on the horizontal floor beyond the toe of glacis. At the toe of the glacis and for some distance above it, therefore, the jet would exert a downward pressure equal to the rate of change of momentum in the vertical direction. The magnitude of this force may be taken as nearly equal to 2/3rd the thickness of the jet at the toe. For determining the range of jump formation, the actual position of the jump on a sloping glacis has been found to be upstream of the calculated position by a distance equal to (D_2-D_1) approximately.

7.3.1.3. In the case of gravity type of floor, the abutments and piers are either independent of the floor, separated from it by means of water tight seals or monolithic with the floor. *If they are to be monolithic it has to be ensured that the thickness of the floor is equal to or more than half the length of the span so that with 45° dispersion, the need for checking for tension does not arise.* However, it is also to be noted that at portions where the floor is fairly thick, some arch action may develop if required and where it is thin, it may act as a beam to the extent it can stand tension when called upon. Both these factors serve as additional safeguard. Hence the designer has to use his discretion in selecting the type of floor, either gravity or raft. Where the gravity floor is of plain concrete, suitable temperature reinforcement will have to be provided.

7.3.1.4. Typical example of design of gravity floor is presented in Appendix 7.2.

7.3.2. *Reinforced Cement Concrete Raft*

Generally, the design of the raft is carried out based on the theory of beams on elastic foundation which takes into account the value of Modulus of Subgrade Reaction (K), span length, total length of raft, etc.

7.3.2.1. However, for small spans upto 6 metres, the raft can be designed as a continuous beam resting on a homogeneous foundation. If the abutments are made independent from the raft, joints with water seals of acceptable standards are to be provided. The raft has to be designed for the moments caused by the worst combination of the following forces : (*i*) uplift (*ii*) soil reaction (*iii*) moments transferred from the abutments and piers and (*iv*) seismic forces if any. For this purpose, the loads transmitted by the abutments and piers are to be assumed to be distributed uniformlly on the foundation. For the design of rafts, any standard text book on Foundation Designs and IS : 2950 (Part - I)—1973 (Code of Practice for Design and Construction of Raft Foundations : Part I - Design) may be referred to.

7.3.2.2. For spans greater than 6 metres, the floor is designed as a finite beam resting on elastic foundation and subjected to concentrated loads and moments at the pier and abutment points.

7.3.2.3. For design of raft using the theory of beams on elastic foundation, Hentenyi's method is usually adopted. Appendix 7.3 is a brief note on the Hetenyi's theory and mentions about the procedure for analysis also. The relevant tables given in Hetenyi's book are to be referred wherever required.

7.3.2.4. Typical calculations of raft using Hetenyi's theory are presented in Appendix 7.4.

7.3.2.5. The overall stability of the barrage/weir in the longitudinal (flow) direction against sliding and overturning is generally checked for pond level condition with water level on the d/s side for minimum flow and gates of adjacent bays closed. For this purpose, one span width including a pier is taken into consideration. All the stabilising forces including the self-wt. of the structure, wt. of water, wt. of road/railway bridges if any, wt. of gates and hoist bridge, etc., are to be estimated. The de-stabilising forces include uplift pressures, water pressure on the closed gates, earth pressures acting on the cut-offs and floor, earthquake forces, etc. While analysing, advantage of the boxed earth below the floor of the barrage and passive pressure of the cut-offs may also be considered wherever applicable. Typical stability calculations are given in Appendix 7.5.

7.4. Design of Peir

In the case of diversion structures, constructed of reinforced cement concrete, piers would be constructed monolithic with the pucca floor. In the case of gravity floor, the piers would generally be made independent of the floor with proper joints and sealing arrangements. Sometimes, they are made monolithic with the floor also as discussed in para 7.8.1.3.

7.4.1. The thickness of the pier is fixed from consideration of (*i*) forces and moments transferred by the pier to the floor/foundation (*ii*) minimum thickness required at the block-outs for the main gate and stop-log grooves and also (*iii*) the weight of the pier required for counter-acting the uplift pressure. The thickness of R.C.C. pier generally varies from 1.5 to 2.5 m.

7.4.2. In the case of raft type of floor, the piers are generally extended upto the full length of the raft to avoid cantilever action at the ends of the floor. However, in the case of gravity floor, the length of the pier can be restricted according to the minimum requirement from considerations of rod/rail bridges, hoist bridge, space required for housing instruments if any, main gate and stoplog grooves, space for storage of stoplogs, adequate length to prevent cross flows occurring which may cause damages to the floor and beyond, etc.

7.4.3. The height of the pier on the upstream side is generally fixed such that the top is above pond level with adequate free board. The height should be fixed from considerations of the requirement of the weigh t of the pier in counteracting the uplift pressure. On the downstream side, the top of the pier should be above the high flood level upto about 3 m beyond the end of the bridges and instrumentation platform if any and thereafter it can be reduced according to low flood levels. Under the road/rail bridges, adequate free board above H F L. needs to be given so that the bearings of the bridges are not affected by the high flood. Adequate allowance should also be made so that the beams of the bridges are not hit by floating debris during high floods. The height is also governed by the requirement of weight to counteract uplift pressures. While fixing the top level of the pier in the main gate portion, it is to be ensured that the bottom of the gate is atleast 1 metre clear of the affluxed high flood level. As already discussed in Para 5.2.5.4., the height of the pier in this portion has to be carefully planned in earthquake regions.

7.4.4. The design of the pier is carried out to resist the worst combination of the following forces and moments :

(*i*) Deal load (*ii*) Live loads due to road/railway bridges (*iii*) Impact and Braking effects of live loads (*iv*) Temperature forces transmitted through bridge bearings (*v*) Dead and Live loads of gates, stoplogs, counterweights and the hoist bridge (*vi*) Breaking effect of gantry crane (*vii*) Buoyancy (*viii*) Differential hydrostatic pressure with one side gate open and the other adjacent gate closed (*ix*) Seismic forces and moments if any and (*x*) Hydrodynamic forces due to seismic conditions if any. The requirement of reinforcement at various levels would be calculated and provided for with suitable curtailments. The reinforcement would be adequately taken into the raft/foundation slab as the case may be and anchored properly.

7.4.5. In the block-out portion, the minimum width of the pier between the blockout on either face would be 600 mm. While calculating the reinforcement in the pier, the increase in the sectional area of the pier in the block-out zone due to the second stage concrete is omitted. Block out-zones would be designed to withstand the concentration of stresses due to the worst combination of the forces and moments mentioned in para 7.4.4. and differential hydrostatic pressure with the gate closed on one side and stoplogs dropped on the other adjacent side. The second stage concrete in the block out zones has to be effectively bounded to the main pier concrete by the provision of adequate dowel bars and secondary reinforcement.

7.4.6. Typical design calculations for pier are given in Appendix 7.6.

7.4.7. In long structures, there would be a number of units/blocks depending on the construction programme and variation in the foundation strata as explained in para 5.2.7. These are separated by double piers with cross cut-off/pile line at the bottom as outlined in para 6.9.3. The thickness of the double pier with a joint in between usually varies from 3 to 5 metres. The thickness, length and height of double pier are governed by the same requirements as those of single pier explained in paras 7.4.1., 7.4.2. end 7.4.3. Each unit of the double pier (two end piers of adjacent bays) would be structurally independent of the other without any transfer of loads and moments from one to the other. The design principles for double pier are the same as those for single pier and each unit of the double pier would be treated as acting completely independent of the other pier.

7.4.8. Under the bridges, properly designed pier caps would be provided satisfying aesthetic requirements also. For spans upto 18 m, the thickness of the pier cap is generally kept not less than 300 mm. The reinforcement in the pier cap would be laid distributed both at the top and bottom in the longitudinal and transverse directions. In addition to this, two layers of mesh reinforcement of 6 mm dia. spaced at 75 mm centre to centre would also be placed under the bearings of the road/rail bridge beams.

7.4.9. *Road Bridges*

The road bridges over the barrages/head regalator (s) are to be designed for the relevant I R.C. class of loading as per the requirements of the projects. The roadwidth including that of footpath, etc., are provided as per requirements. Relevant Indian Road Congress Codes may be referred for the details of loading and other design norms, etc.

7.5. Design of Divide Wall

The functions and layout of divide wall have been explained in para 5.2.6. and its sub-paras. The hydraulic designs of the same are outlined in para 6.11. The upstream divide wall would be designed to resist the differential head on account of the velocity of flow in the pocket due to the gates adjacent to the divide wall on either side being kept in open and closed condition. The downstream divide wall would be designed to resist the moments due to the differential head caused by the closure of gate on one side and opening of gate on the other side. While the range of differential heads for the designs would be indicated by hydraulic model tests wherever conducted, for preliminary *desgins, usually a differential head of about 1.5 to 2 m is assumed.* Under seismic conditions, differential head due to wave action is not considered as the likelihood of earthquake and high wind occurring simultaneously is rather remote. Dynamic increase in water pressure is calculated from the formula,

$p{=}875\ \alpha_h\ \sqrt{H.y}$ where

p=hydrodynamic pressure in kg/m^2

α_h=design horizontal seismic coefficient

H=height of water surface from the level of deepest scour in metres, and

y=depth of the section below the water surface in metres.

Reference may be made to IS : 1893—1975—Criteria for Earthquake Resistant Design of Structures (Third Revision). The divide wall will also be checked for their overall stability considering the forces and moments due to self weight, water, ice (wherever applicable), uplift, earthquake, etc., acting on the wall.

7.6. Design of Abutments

The general considerations for the layout of abutments are discussed in para 5.2.8. The top width of the abutment in each block is fixed as per the requirements due to the loads and moments and minimum width required for block-outs of main gate and stoplog grooves, bridge bearings, gate trestle foundation, etc. The minimum width of abutments clear of the block-outs and behind bearing niches would be kept as 600 mm. Generally the minimum top width of the abutment in the upstream and downstream blocks iskept as 600 mm and the same in the gate and road/rail bridge blocks is kept as 1250 and 1400 mm respectively.

7.6.1. While designing the various abutment blocks, worst combination of the following forces and moments pertaining to the block under consideration is taken into account : (*i*) Dead load (*ii*) Live load due to moving traffic over the bridges (*iii*) Impact and braking effect of live loads (*iv*) Temperature forces transmitted through bridge bearings (*v*) Dead and Live loads of gate and gate bridge (*vi*) Braking effect of gantry crane (*vii*) Earth pressure, live load surcharge and saturation pressure (*viii*) uplift (*ix*) Seismic forces and moments if any (*x*) Hydro-dynamic forces due to seismic conditions if any. The abutment is generally designed as a retaining wall, either as cantilever type or counterfort type as the case may be. The reinforcement will be provided accordingly with suitable curtailment or spacing wherever necessary.

7.6.2. The abutment section, designed on the guide lines given above, has to be checked for safety against allowable bearing pressure, overturning and sliding. The allowable bearing pressure would be worked out by field and laboratory tests following standard procedures. The factor of safety against overturning should not be less than 2.0 under normal conditions of loading and not less than 1.5 under seismic conditions. While calculating the sliding factors, the value of the coefficient of friction, determined by field investigations are to be used. If this is not available, the following values of coefficient of friction may be used :—

(*i*) Coarse sandy soil containing no silt or clay —0.55

(*ii*) Coarse sandy soil containing silt—0.45.

Also, wherever concrete/masonry or sheet pile cut-offs have been provided 50 percent of the passive pressure developed below the level of the deepest scour may be considered to the extent of the structural strength of the cut off provided while calculating the counteracting forces against sliding.

7.6.3. Typical design calculations for one of the blocks of the abutment are presented in Appendix 7.7.

7.7. Design of Flank Wall

The general considerations for the layout of flank walls are discussed in para 5.2.9. The top width of the flank wall is generally kept not less than 600 mm. The top width of the flank wall made up of c.c. blocks varies with the size of the c.c. blocks and generally it is 1500 mm. The flank wall is also designed as an earth retaining structure for the worst combination of the following forces and moments :—(*i*) Dead loads (*ii*) Earth pressure, live load surcharge and saturation pressures (*iii*) Uplift (*iv*) Seismic forces and moments if any and (*v*) Hydrodynamic forces due to seismic condition if any. Just like the abutment, flank wall also has to be checked for safety against allowable bearing presssure, overturning and sliding. The earth pressures acting, behind the wall would be calculated by any of the standard methods available.

7.8. Design of Return Wall

The general considerations for the layout of return walls are discussed in para 5.2.10. The other design considerations like top width, forces and moments, etc., are similar to those of flank wall, outlined in para 7.7 above.

7.9. In the foregoing paras, general guide lines for the structural design of the main components of the diversion structure have been set out. These, in addition to the other guide lines discussed in some of the other Chapters on head regulator, silt exclusion, navigational facilities, fish pass and river training works can be modified by the designer to suit the conditions obtaining at the site. But safety and economy are always to be ensured and constructional aspects should always be considered while fixing up the reinforcement, piles, etc.

APPENDIX 7.1

Typical Sheet Pile Calculations

Calculations presented are in British units.

Maximum estimated intensity of discharge

$q_1 = 409$ cusec/ft.

Intensity of discharge with 10 percent concentration

$= 450$ cusec/ft.

Anticipated scour level

$= +10.70$ ft.

$\phi = 30°$

Coeff of active earth pressure

$= \frac{1}{3}$

Coeff. of passive earth pressure

$= 3$

Surcharge pressure

$= \frac{1}{3} \times 4.5\ (150 - 62.5) = 130$ lb

Active earth pressure (submerged soil) at $+10.70$

$= \frac{1}{3} \times 13.8 \times 62.5 = 288$ lb

Net pressure at $+10.70 = 130 + 288$

$= 418$ say 420 lb.

Rate of change of pressure $K = (3 - 0.33) \times 62.5$

$= 167$ lb.

Hence $\quad a = 420/167 = 2.52$

Considering the pile to be simply supported beam between A and B,

$$R_A + R_B = (130 \times 13.8) + \left(\frac{1}{2} \times 290 \times 13.8\right) + \left(\frac{2.52 \times 420}{2}\right)$$

$$= 1794 \times 2000 + 527 = 4330 \text{ lb.}$$

$$R_B \times 16.32 = (1794 \times 6.9) + (2000 \times 9.2) + (527 \times 14.64)$$

$$= 12400 + 18400 + 7750$$

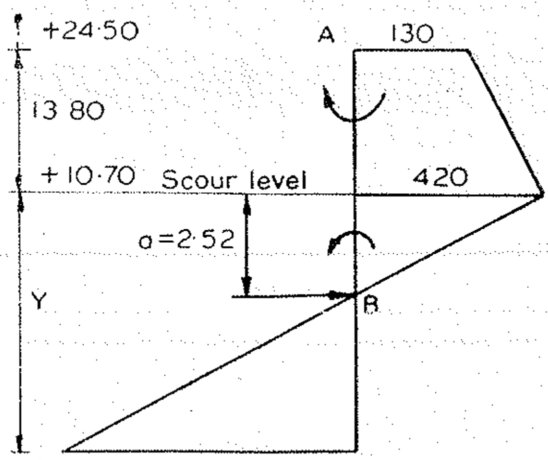


FIGURE 7.1

$=38550$

$R_B = 2360$ lb $R_A = 4330 - 2360$

$= 1970$ lb.

Let the minimum penetration required be y ft. below scour level.

Taking moment about the bottom of sheet pile,

$$R_B\,(y-a) \quad K = \frac{(y-a}{2} \times (y-a)\frac{(y-a)}{3}$$

$$R_B = K\,\frac{(y-a)^2}{6}$$

$$(y-a) = \sqrt{\frac{6R_B}{K}} = \sqrt{\frac{6 \times 2360}{167}} = 9.24$$

$$y = 9.24 + 2.52 = 11.76 \text{ ft.}$$

Allowing 20 percent extra embedment in sandy material (as suggested in Yawata sheet piling literature)

$$y_{reqd} = 11.76 \times 1.2 = 14.1$$

Scour Level $= +10.70$

$\therefore$ Bottom of sheet pile $= 10.7 - 24.1 = -3.4$ or say -3.5

To find M_{max} BM. Let Max B.M. be x below A.

$$M_{max} = 1970\,x - 130 \times x \times \frac{x}{2}$$

$$- 290 \times \frac{x}{13.8} \times x \times \frac{x}{3}$$

$$= 1970\,x - 65x^2 - 3.5^3$$

$$\frac{dy}{dx} = 1970 - 130x - 10.5x^2 = 0$$

$$x^2 + 12.4x = 187.5 = 0$$

$$x = \frac{-12.4 \pm \sqrt{154 + 750}}{2} = 8{\cdot}8^1$$

Hence maximum moment

$$= 1970 \times 8{\cdot}8 - 65 \times 8{\cdot}8^2 - 3{\cdot}5 \times 8{\cdot}8^3$$

$$= 17300 - 5040 - 2380 = 9880 \text{ ft. lb.}$$

$$= 118500 \text{ in. lb.}$$

Assuming $f = 16000$ psi

$$Z_{reqd} = \frac{118500}{16000} = 7{\cdot}4 \text{ in}^3\text{/ft run of wall}$$

of pile line.

APPENDIX 7.2

Typical Design of Gravity Floors

Calculations are presented for a gravity type of floor with the following parameters :

Pond level =328·66 m

HFL for an anticipated flood of 17000 cumecs =331·63 m

Rterogression at High flood=0.61 m

Retrogression at Pond level=1.07 m

The % pressure have been worked out as follows :

Upstream pile line φ_E corrected=73.47%

Intermediate pile line φ_E corrected=57·8%

φ_C corrected=41·26%

Downstream pile line φ_E corrected=33·19%

The levels of the hydraulic gradient lines at key points under different flow conditions are given in Table 7.1.

The table 7.2 has been prepared for plotting the hydraulic jump for the High Flood and Pond level conditions.

The thickness of the floor is determined by dividing the unbalanced head given by the ordinate between the hydraulic gradient line and the corresponding profile of water by (G-1) where G is the specific gravity of concrete. To reduce the floor thickness, advantage has been taken of the shear strength of floor and the dynamic downward thrust of the jet near the toe of glacis. Instead of taking the max. ordinate, an average over a length of 1.5 m has been taken and dynamic thrust is taken as $\frac{2}{3}$ jet thickness. sp. gravity of concrete=2.3 Table 7.3

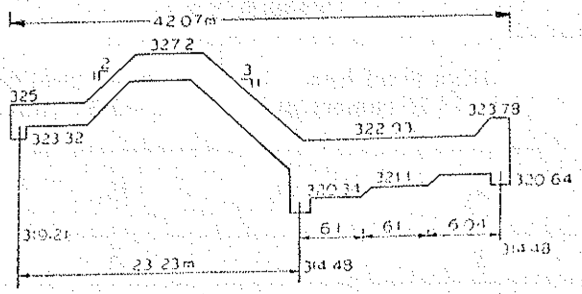


FIGURE 7.2 :

TABLE 7.1

Condition	D/S WL Datum	U/S W.L.	Head	Ht/Elev. of Sub-soil H.G. line above datum								
				U/S pile line			Inter pile line			D/S pile line		
				φ_E	φ_D	φ_C	φ_{E_1}	φ_{D_1}	φ_{C_1}	φ_{E_2}	φ_{D_2}	φ_{C_2}
				100%	77.5	73.47	57.8	47	41.26	33.19	26.5	0
No flow Maximum	322 93	328.66	Height 5.73	5.73	4.44	4.21	3.31	2'7	2 38	1.9	1.52	0
Static Head			Elevation	328.66	327.36	327.13	326.24	325.63	325 38	329.13	324.45	322.93
High Flood	331'02	332.32	Height 1.3	1.3	1.01	0'95	0.75	0.61	0.54	0.43	0.35	0
			Elevation	332.32	332.03	331.97	331.77	331.63	331.56	331.45	331.37	331.02
Flow at Pond Level	325'5	328.66	Height 3.16	3.16	2.45	2.32	1'82	1'48	1.31	1.05	0'84	0
			Elevation	328.66	327.95	327.82	327'32	326.98	326.81	326.55	326.34	325.5

The hydraulic gradient lines in accordance with the above values have been plotted in the sketch.

TABLE 7.2

Distance from d/s end of crest	R.L. of glacis	High flood flow $q=24.26$ cumecs/m		Pond level flow $q=3.35$ cumecs/h		Remarks
		E_{f_1}	D_1	E_{f_1}	D_1	
1	2	3	4	5	6	7
1.83	326.59	6.13	3.2	2.07	0.64	
3.66	325.98	6.74	2.74	2.68	0.52	U/S TEL for High flood
5.49	325.37	7.35	2.5	3 29	0.43	332.72
7.32	324.76	7.96	2.29	3.9	0.4	
8.63	324.32 Jump Location for HF	8.4	2.23	—	—	Crest level 327.2
9.15	324.15	—	—	4.51	0 38	Slope of d/s
10.98	323.54	—	—	5.12	0.35	glacis 3 : 1
12.67	322.97	—	—	5.69	0.34	

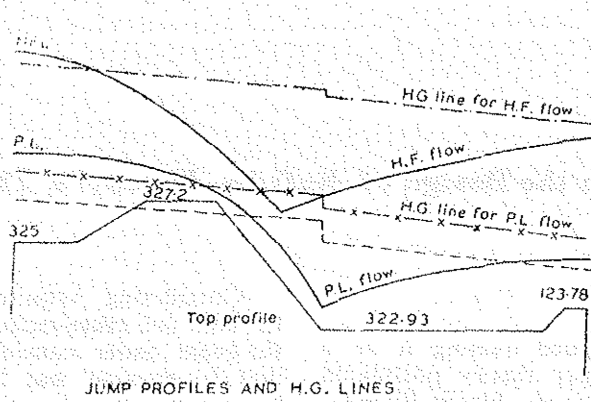


FIGURE 7.3 :Jump Profiles and H.G. Lines.

No.	Distance from d/s end of crest	Average Ordinate m		⅔ Jet thickness m		Net Ordinate m	Thickness reqd. m	Thicknss provided m
1.	8.63	4.88 5.34 5.03	5.08	1.68 (av.)		3.40	2.62	2.6
2.	12.8	3.81 4.12 3.2	3.71	0.31 0.31 0.61	0.41	3.3	2.54	2.6
3.	18.9	2.29 2.26 2.21	2.25	—		2.25	1.73	1.83
4.	25	2.06		—		2.06	1.59	1.68

APPENDIX 7.3

Note on the Design of Raft by Hetenyi's Theory

Introduction

The raft of a Barrage is designed treating it as a beam on elastic foundation as per theory postulated by M. Hetenyi in his bo k 'Beams on Elastic Foundations' (published by the 'University of Michigan Press' of U.S.A).

Hetenyis's Theory

The theory is based on the assumption that the reaction force of the foundation are proportional to the deflection of the beam at that point. This assumption is represented in the figure 7.4 :

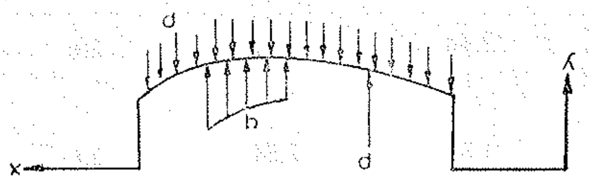


FIGURE 7.4.

The above assumption implies that the supporting medium (foundation soil) is elastic conforming to Hooke's law. This is mathematically represented by $p=Ky$. The 'Elasticity' can be characterized by the force distributed over a unit area causing unit deflection. This is represented by Ko. kg/cm^2 and called 'modulus of the foundation' or 'Modulus of Subgrade reaction'.

Assuming that a beam under consideration has a uniform cross-section and that 'b' is its constant width, which is supported on the foundation, unit deflection of this beam will cause reaction 'bKo' in the foundation. Consequently at a point where the deflection is 'y' the intensity of distributed reaction (per unit length of the beam) will be

$$p\ kg/cm^2=b\ Ko\ y)$$

The symbol K kg/cm^2 could be used for $b\times Ko$ remembering that K includes the effect of the width of the beam and will be numerically equal to Ko when the width of the beam is unity.

Considering infinitely a small element enclosed between two cross-sections a distance 'dx' apart on the beam under consideration, it may be presumed that this element was taken from a portion of the beam acted upon by a distributed loading q kg/cm. The forces acted upon on the element are indicated in Figure 7.5.

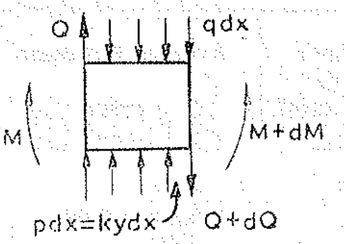


FIGURE 7.5.

The upward acting shearing force 'Q' to the left of the cross-section is considered positive, as in the corresponding bending moment 'M' which is a clockwise moment acting from the left on the element. Considering the equilibrium of the element, it is seen that the summation on the vertical forces give

$$Q-(Q+dQ)+Kydx-qdx=0$$

whence

$$\frac{dQ}{dx}=Ky-q \qquad ...(1)$$

Making use of the relation $Q=\frac{dM}{dx}$ it can be written

$$\frac{dQ}{dx}=\frac{d^2M}{dx^2}=Ky-q \qquad ...(2)$$

From the knowledge of theory of structures the differential equation of a beam in bending can be written as

$$EI\left(\frac{d^2y}{dx^2}\right)=-M$$

Differentiating the above Expression,

$$EI\frac{d^4y}{dx^4} = -\frac{d^2M}{dx^2} \quad ...(3)$$

Therefore from (2) and (3)

$$EI\frac{d^4y}{dx^4} = -Ky+q \quad ...(I)$$

This is the differential equation for the deflection curve of a beam supported on an Elastic foundation. Along the unloaded parts of the beam, where no distributed load is acting ($q=0$), the equation would reduce to

$$EI\frac{d^4y}{dx4} = -Ky \quad ...(II)$$

It will be sufficient to consider the general solution (II) from which solution can be obtained also for cases implied in (I) by adding to it a particular inlegral corresponding to q in (I)

Substituting $y=e^{mx}$ in (II) the characteristic equation $m^4-\frac{K}{EI}$ is obtained which has the roots

$$m_1=-m_3=4\sqrt{\frac{K}{4EI}}\ (1+i)=\lambda\ (1+i)$$

$$m_2=-m_4=4\sqrt{\frac{K}{4KI}}\ (-1+i)=\lambda(-1+i)$$

where $\lambda=4\sqrt{\frac{K}{4EI}}$

Then the general solution of (II) takes the form

$$y=A_1e^{m1}x+A_2e^{m2x}+A_3e^{m3x}+A_4e^{m4x} \quad ...(III)$$

using

$$e^{i\lambda x}=\cos \lambda x+i \sin \lambda x$$

$$e^{-i\lambda x}=\cos \lambda x-i \sin \lambda x$$

and introducing the new constants

C_1, C_2, C_3 and C_4, Where

$(A_1+A_4)=C_1$ $\qquad i\ (A_1-A_4)=C_2$

$(A_2+A_3)=C_2$ $\qquad i\ (A_2-A_3)=C_4$

(III) can be written in a more convenient form as

$$y=e^{\lambda x}\ (C_1 \cos \lambda x+C_2 \text{ Sin } \lambda x)+e^{-\lambda x}\ (C_3 \cos \lambda x+C_4 \text{ Sin } \lambda x) \quad ...(IV)$$

Here λ includes the flextural rigidity of the beam as well as the elasticity of the supporting medium (foundation soil) and is an important factor influencing the shape of the elastic line. For this reason the factor λ is called the 'Characteristic, of the system and its dimension is length^{-1}. The term $1/\lambda$ is frequently referred to as the characteristic length. Consequently 'λx' will be an absolute number.

Expression (IV) represents the general solution for the deflection line of a beam supported on Elastic foundation and subjected to transverse bending force. By successive differentiation, Expressions for slope of the deflection curve, Bending moment and shearing force can be obtained.

However, Hetenyi had already derived expressions from the above general Equation in his book for different loading conditions which can be conveniently adopted by the designer to derive an equation for a particular loading from the general Equation IV.

Modulus of Sub-Grade Reaction (K)

The value of the sub-grade reaction is to be determined in accordance with the IS : 1888—Method of Load Tests on Soils.

In the absence of detailed soil investigations the values of sub-grade reaction to be adopted are given in IS 2950—1973 Code of Practice for Design and construction of Raft foundations. These are reproduced below for ready reference.

Modulus of Subgrade Reaction for Cohesionless Soils.

Relative density	Standard Penetration Test Value (N) (blows for 30 cm.)	Moduls of sub-grade reaction K_5 in kg/cm^2	
		For dry or Moist State	For Submerged State
Loose	<10	1·5	0.9
Medium	10 to <30	4.7	2.9
Dense	30 and over	18.0	10.8

Modulus of Subgrade Reaction for Cohesive Soils.

Consistency	Unconfiend compressive Strength kg/cm²	Modulus of Subgrade reaction K_6 in kg/cm²
Stiff	1 to <2	2.7
Very Stiff	2 to <4	5.4
Hard	4 and over	10.8

The above values are based on the assumption that the subgrade reaction does not exceed half the ultimate bearing capacity.

The values given above are applicable for a square plate 30 cm × 30 cm or beams 30 cm wide. For finding the values of subgrade reaction K corresponding to sizes and shapes different from those of the following relationships shall be used.

Effect of Size

$$K = K_s \left(\frac{B+30}{2B} \right)^2 \text{ for cohesionless soil.}$$

$$K = x \frac{K'}{B} \text{ for cohesive soil.}$$

where

K = Modulus of subgrade reaction for footing of width B cm.

K_s = Modulus of subgrade reaction for a square plate 30 × 30 cm.

K' = Modulus of subgrade reaction for footing of width × cm.

Effect of Shape

For cohesive soils

$$K_1 = K_s \frac{2}{3} \left(1 + \frac{B}{2L} \right)$$

Where

K_1 = Modulus of subgrade reaction for a rectangular footing having a length L and width B.

K_2 = Modulus of subgrade reaction for a square footing of side B.

The effect of shape is negligible in the case of footing on cohesionless material.

Loads

(*a*) *Gate, Road and Railways Bridges.* These loads include the dead weight of the Bridges and the anticipated max live loads on the Bridges. These loads are transferred to the raft through the piers dispersing itself at 45° to the raft level.

Due to the Braking and tractive effect of the live loads Horizontal forces act at the bearings and are transferred to the raft as moments at a dispersion of 45° to the raft level.

In the event of Earthquake condition Inertia and impact forces are also to be considered.

(*b*) *Dead weight of piers.* The weight of the pier is considered on the raft as a concentrated load at the pier location.

Inertia force is also to be accounted in the event of earthquake condition.

(*c*) *Differential Head forces.* Due to differential water levels on either side of the Piers a moment is created and transferred to the raft. This condition is more obtained when Hydraulic Jumps takes places in one bay while the gates of the adjacent bays are closed. Differential water levels also occur when the flow is not axial and cross currents develop.

In the earthquake condition Hydro-dynamic forces are caused on the piers and transferred to the raft as moments.

(*d*) *Uplift force and self weight of raft.* The uplift force is countered by the self weight of raft. In case the uplift force is more than the weight of the raft the residue is known as 'unbalanced uplift' which acts upwards as a uniformly distributed load on the raft while the raft is held down by the piers.

A force diagram indicating the above loads would be as shown in Figure 7.6.

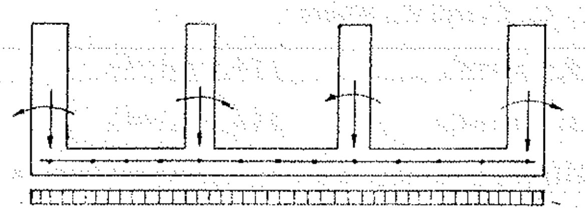


FIGURE 7.6.

Analysis

The first step would be to compute the different loads that would be transferred to the raft.

Then the induced moments in the raft due to these loads are calculated along the length of the raft at selected stations. Normally the selected stations would be below the piers. Quarter spans and mid spans. The induced moments in the raft due to the loads mentioned in last para barring the 'unbalanced uplift' are calculated utilising the equations developed by Hetenyi in his book for different type of loads *viz.*, concentrated loads, moments, udl etc. The analysis of the induced moments due to the 'unbalanced uplift' is to be done treating the raft as a continuous beam spanning the piers by the method of moment distribution.

Then the induced moments at a station due to these different loads are superimposed and the net value computed for the station. This would be done for all the stations so chosen. Finally, having computed the net induced moments at different stations the steel required at each such station is computed and provided.

APPENDIX 7.4

Typical Design of Raft

The pucca floor of the barrage (in the example given below) has been designed as beams on elastic foundation. The length of raft has been divided into fair zones depending on the loading intensities. The different zones are in the regions of upstream floor, gate and stoplog, downstream glacis and downstream cistern. The width of the rafts are taken between the expansion joints as follows :

Left undersluice bay	1 to 3	32·97 m
Spillway bay	4 to 8	62 60 m
Spillway bay	9 to 12	50·50 m

The moments on account of pier loads have been calculated as per Hetenyi's method (Beams on elastic foundations). In addition moments caused on piers due to differential head of water also have been taken into account in determining the total design moments. 20 percent extra reinforcement over the calculated values have been provided to take care of the limitations in the assumptions and bending moments calculated. The raft has been designed for the following conditions and the governing values adopted for reinforcement purposes.

(*i*) Pond level condition

(*ii*) Hydraulic jump condition

(*iii*) Downstream dry.

Typical calculations are given in the succeeding pages for the undersluice raft in zone 3 (D/S glacis region) on the left side. The required reinforcement has been suitably adjusted keeping the construction points of view also.

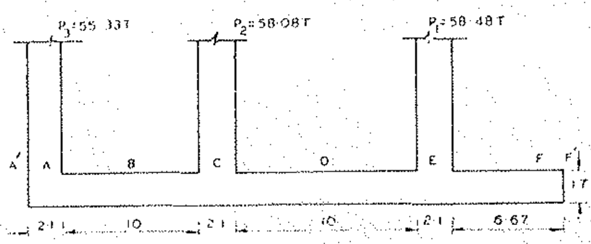


FIGURE 7.7

$K = 100$ Tonnes/ft^2/ft$=3530$ Tonnes/m^2/m

$E_{concrete} = 1800\sqrt{\sigma_{co}}$ kg/cm^2

$=18000\sqrt{150}$ kg/cm^2

$=2{\cdot}2\times10^6=$tonne/m^2

$= \frac{1}{12}bd^3 = \frac{1}{12}\times1\times1.7^3 = 0{,}407$ m^4

$$\lambda = \sqrt[4]{\frac{K}{4\,EI}} = \sqrt[4]{\frac{3530}{4\times2{\cdot}2\times10^6\times{\cdot}407}}$$

$=0{\cdot}1775$ m^{-1}

$\frac{1}{\lambda} = 5{\cdot}55$

$\lambda_L = 0{\cdot}17775\times30>\pi$

Hence this will be treated as long beam

Unit Load at end A^1

Moment $M = \frac{1}{\lambda}\,B_{\lambda x}$.

TABLE 7.4

	A′	A	B	C	D	E	F
x	0	1.05	7.1	13.15	19.20	25.25	29.65
λ_x	0	0.185	1.26	2.34	3.40	4.50	5.25
$B_{\lambda x}$	0	0.1528	0.2807	0.0880	—0.0032	—8.0128	—0.0065
$-\frac{1}{\lambda} B_{\lambda x}$	0	—0.855	−1·56	—0.495	0.0178	—0.0720	0.036

$a = 13{\cdot}15$

$\lambda = 0\ 1775 \qquad 4\lambda = {\cdot}7100$

$\lambda_a = 2.33$

$$M = \frac{1}{4\lambda}\ [\alpha C\lambda x - 2\beta\ D\lambda x + C\lambda,\ |\ a-x\ |\ ]$$

where $\alpha = C\lambda a + 2D\lambda a$

$= -0{\cdot}1376 + 2\ (-0{\cdot}0670)$

$= -0{\cdot}1376 - 0{\cdot}0134$

$= -0{\cdot}1510$

$\beta = C\lambda a + D\lambda a$

$= -0{\cdot}1376 - 0{\cdot}0670$

$= -0{\cdot}2046$

Also $\frac{1}{4\lambda} = 1{\cdot}40$

$\frac{\alpha}{4\lambda} = 0\ 212$

$\frac{2\beta}{4\lambda} = -0{\cdot}575$

$M = 0{\cdot}212\ C\lambda x + 0{\cdot}575\ D\lambda x + 1.40\ C\lambda\ |\ a-x\ |$ (Table 7.5)

TABLE 7.5

S. No.		A	B	C	D	E	F
1.	x	1.05	7.1	13.15	19.2	25.25	29.65
2.	λx	0.185	1.26	2.34	3.40	4.45	5.25
3.	$\mid a-x \mid$	12 10	6.05	0	6.05	12.10	16.50
4.	$\lambda \mid a-x \mid$	2.15	1.07	0	1.07	2.15	2.93
5.	$C\lambda x$	0 664	—0.1560	—0 1372	—0.0238	0.008	0 0073
6.	$D\lambda x$	0.8170	0.1399	—0.0611	—0.0323	0.0030	0.0027
7.	$C\lambda \mid a-x \mid$	− 0.1613	—0.1362	1.0	—0.1362	−0.1613	—0.0634
8.	$0.212\ C\lambda a$	—0.141	0.033	0.029	0.0055	—0.0017	—0 0015
9.	$0.575\ D\lambda x$	0.470	0.0805	—0.039	—0.0186	—0 0017	0.0015
10.	$1.4\ x\ C\lambda \mid a-x \mid$	—0.225	—0.1900	−1.40	—0.1900	—0.225	—0.088
11.	Total of 8+9+10	+0.110	—0.077	1.390	—0.2031	—0.2284	—0.088

Unit Load at E

$$M=\frac{1}{4\lambda}\left[\alpha C\lambda x-2\beta D\lambda x+C\lambda \mid x-a-\right]$$

$a=25.25$

$\lambda=0.1775$

$\lambda a=4.48$

$\alpha=C\lambda a+2D\lambda a=0.0084-2\times0.0025$
$=0.0034$

$$\text{Force}=\frac{\omega H^2}{2}=\frac{1\times3{\cdot}44^2}{2}=6\ T$$

Moments M=Force$\times2.31=6\times2.31=14$ T.M.

At points A, C, E, moment$=\pm14$ T.M.

At points B, D, F, moment$=\pm10$ T.M.

Final moments (Table 7.6)

A_t reqd.

in cm²	−32	−59	34	−16	56	7

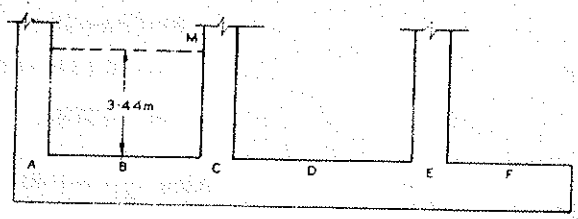


FIGURE 7.8

$\beta=C\lambda a+D\lambda a=0.0084-0.0025=0.0059$

$$\frac{\alpha}{4\lambda}=\frac{0.0034}{4\times0.1775}=0.00475$$

$$\frac{2\beta}{4\lambda}=\frac{2\times0.0059}{4\times0.1775}=0.166$$

$M=0.00475\ C\lambda x-0.166\ D\lambda x+0.140\ C\lambda \mid x-a\mid$

Moments due to differential head is given in Figure 7.8.

$$A_t=\frac{M}{tjd}=\frac{M\times1000\times100}{1400\times0.87\times160}\ \text{sq cm}$$

$a=25.25$

$\lambda=0{\cdot}1775$ (Table 7.7)

Multiplying the unit moments by respective loads v. The first table by 55.33 and second table by 58·08 and third by 57.48

TABLE 7.6

	A	B	C	D	E	F
By moment distribution	0.49	−11.3	9.73	1.76	26.60	5.0
By Hetenyi	−47.53	−94.78	42.23	−21.15	70.58	−2.94
By different head	−14.0	−10.0	14.0	−10.0	14.0	10.0
Total	−61.04	−116.08	65.96	−29.39	111.18	12.06

TABLE 7.7

S. No.		A	B	C	D	E	F
1.	x	1.05	7.1	13.15	19.2	25.25	29 65
2.	λx	0.185	1.26	2.34	3.40	4.50	5.25
3.	$\mid x-a \mid$	24.20	18.15	12.10	6.05	0	4.40
4.	$\lambda \mid x-a \mid$	4.30	3.22	2.15	1.07	0	0.78
5.	$C\lambda x$	0.664	—0.156	—0.1372	—0.0238	0.008	0.0073
6.	$D\lambda x$	0.817	0.1399	—0.0671	—0.0323	—0.003	0.0027
7.	$C\lambda \mid x-a \mid$	0.0070	0.0367	—0.1625	—0.1313	1	0.0035
8.	$0.0047\, C\lambda x$	0.0036	—0.0007	—0.0006	0	0	0
9.	$-0.166 D\lambda x$	—0.136	—0.0144	—0.112	0.0053	—0.0005	—0.0005
10.	$1.4\, C\, \lambda \mid x-a \mid$	0.0118	—0.0510	—0.2260	—0.1890	1.40	0.0048
11.	Total of 8+9+10	—0.1206	—0.0161	—0.2154	—0.1837	1.4005	0.0043

A	*B*	*E*	*F*
—47.3	—86.5	+ 3 93	1.99
+ 6.75	— 4.48	—13.35	— 5.18
— 6.98	— 3.80	+80.00	+10.25
47.53	—94.78	+70.58	— 2.94

C	*D*
—27.4	+ 0.98
+81.0	—11.6
—12.37	—10.53
+42.23	—21.15

Moment Distribution

Average uplift $= \dfrac{61+3.82}{2} = 4.96$

(from uplift pr. calculations)

Wt. of slab $= 1.7 \times 2.4 = 4.08$

$\therefore$ Net uplift $= 4.96 - 4.08 = 0.88$ say 0.9 T/m^2.

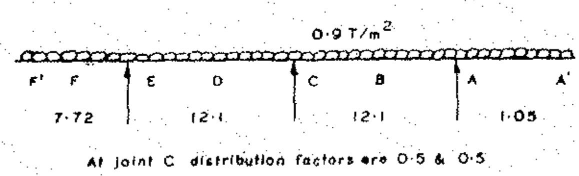


FIGURE 7.9

Fixed end moments

$$M_{EC}=\frac{-wl^2}{12}=\frac{-0{\cdot}9\times 12{\cdot}1^2}{12}=-11$$

$$M_{CE}=+11\ \&\ -11=M_{CA}$$

$$M_{AC}=+11$$

Moment at E due to *udl* on EF'

$$=\frac{0{\cdot}9\times 7{\cdot}72^2}{2}=26{\cdot}6$$

Moment at A due to *udl* on $AA'=\frac{0{\cdot}9\times 1{\cdot}0^2}{2}$

$$=-0{\cdot}49$$

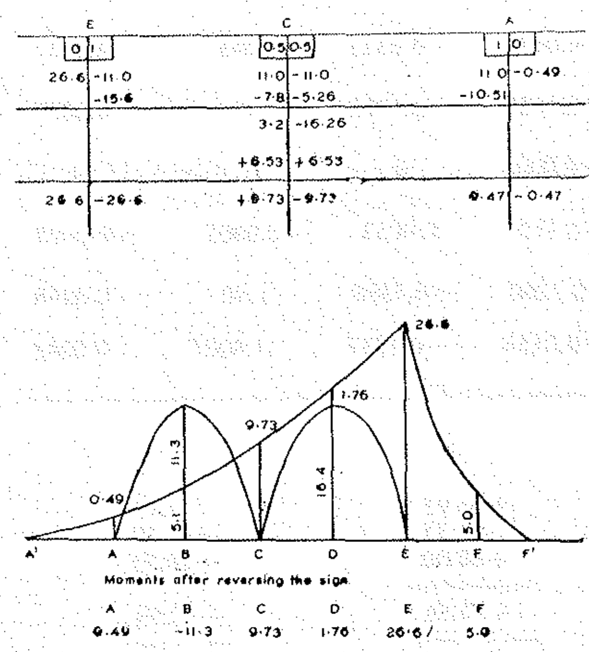


FIGURE 7.10.

Sagging moments $=\frac{wl^2}{8}=\frac{0{\cdot}9\times 12{\cdot}1^2}{8}=16{\cdot}4$

Moment at F due to *udl* on $EF'=\frac{wl^2}{2}$

$$=\frac{0{\cdot}9\times 3{\cdot}33^2}{2}=5$$

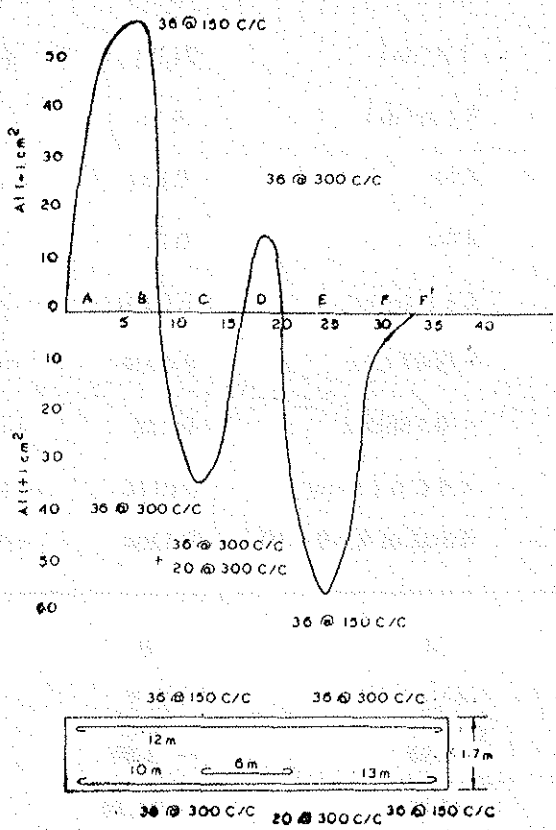


FIGURE 7.11.

APPENDIX 7.5

Typical Stability Calculations

Typical Stability Calculations

Taking a typical section of spillway as shown in figure 7.12 the various forces have been estimated.

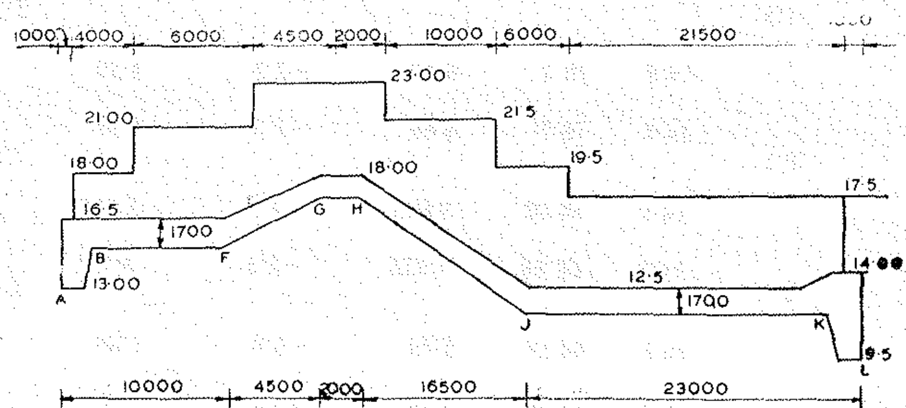


FIGURE 7.12

(1) Area of section of raft $= 1{\cdot}8\,\dfrac{(1+1{\cdot}5)}{2} + 10\times1{\cdot}7$

$+4{\cdot}74\times1{\cdot}7+2\times1{\cdot}7+17{\cdot}39\times1{\cdot}7$

$+23\times1{\cdot}7+1{\cdot}5\dfrac{(1{\cdot}5+3)}{2}+1{\cdot}3\dfrac{(1+1{\cdot}5)}{2}$

$=2{\cdot}25+57{\cdot}13\times1{\cdot}7+3{\cdot}375+1{\cdot}625$

$=7{\cdot}25+97{\cdot}12=104{\cdot}37$ sq. m.

Wt. of raft/bay $=104{\cdot}37\times12{\cdot}1\times2{\cdot}4=3031$ Tonnes

Clear span $=10$ m & width of pier $=2{\cdot}1$ m.

Wt. of friction blocks/bay $=(4\times1\times1{\cdot}25\times1$

$+4\times1\times1{\cdot}25\times1{\cdot}25)\;2{\cdot}4$

$=(5+6{\cdot}25)\;2{\cdot}4=27$ Tonnes.

Wt. of raft+friction blocks/bay $=3058$ Tonnes.

(2) Wt of Pier→Area $=261{\cdot}65$ sq. m.

Wt. $=261{\cdot}65\times2{\cdot}1\times2{\cdot}4=1318{\cdot}7$ say 1320 Tonnes.

(3) Wt. of water (D/S dry) $=64\times10\times1+(1\times2{\cdot}1\times4{\cdot}5\times1)=649$ Tonnes.

(4) Uplift Pressure

ϕ_C Corrected $=80{\cdot}4\%$

ϕ_E Corrected $=18\%$

Slope $=\dfrac{80{\cdot}4-18}{56}=1{\cdot}114$ (of Pr. diagram)

$\phi_B=54{\cdot}5\times1{\cdot}114+18=78{\cdot}713\%$

$\phi_F=46\times1{\cdot}114+18=69{\cdot}244\%$

Slope Correction $= \dfrac{4.5\times4.5}{56} = 0.361\%$

ϕ_F Corrected $= 69.244 - 0.361 = 68.883\%$

$\phi_G = 1.114\times41.5+18 = 64.231\%$

ϕ_G Corrected $= 64.231 + 0.361 = 64.592\%$

$\phi_H = 1.114\times39.5+18 = 62.003$

Slope Correction $= \dfrac{4.5\times16.5}{56} = 1.33\%$

ϕ_H Corrected $= 62.003 + 1.33 = 63.333\%$

$\phi_J = 1.114\times23+18 = 43.622\%$

ϕ_J Corrected $= 43.622 - 1.33 = 42.292\%$

$\phi_K = 1.114\times1+18 = 19.114\%$

Pond level

Head of water $= 21 - 12.5 = 8.5$ m.

Point	Distance	Width	R.L.	%Pr.	Uplift H	TWL-RL	Total	Average	Uplift
A	0		13·0	80·4	6·834	−0·5	6·334		
		1·5						5·362	8·043
B	1·5		14·8	78·7	6·69	−2·3	4·39		
B	1·5		14·8	78·7	6·69	−2·3	4·39		
		8·5						3·972	33·766
F	10		14·8	68·88	5·855	−2·3	3·555		
F	10		14·8	68·88	5·855	−2·3	3·555		
		4·5						2·623	11·80
G	14·5		16·3	64·59	5·49	−3·8	1·69		
G	14·5		16·3	64·59	5·49	−3·8	1·69		
		2·0						1·637	3·273
H	16·5		16·3	63·33	5·383	−3·8	1·583		
H	16·5		16·3	63·33	5·383	−3·8	1·583		
		16·5						3·439	56·744
J	33·0		10·8	42·29	3·595	+1·7	5·295		
J	33·0		10·8	42·29	3·595	+1·7	5·295		
		21·5						4·31	92 665
K	54·5		10·8	19 11	1·625	+1·7	3·325		
K	54·5		10·8	19·11	1·625	+1·7	3·325		
		1·5						3·928	5·891
L	56·0		9·5	18	1.53	+3·0	4·53		
							Total		212·182

Total uplift/bay $= 212.182\times12.1\times1 = 2567.4$ say 2580 Tonnes allowing for cut off width

(5) Boxed Sand.

Area of section $=\frac{2+11}{2}\times 1\cdot 5+\frac{19\cdot 5+25\cdot 4}{2}\times 1\cdot 8$

$=50\cdot 16$ sq. m.

Weight $=50\cdot 16\times 12\cdot 1\times 1=606\cdot 936$ say

607 Tonnes.

(6) Horizontal Water Pressure

Location	Head in metre Max	Min	Average	Height m	Length m	Force in Tonnes →	←
U/S Cut off	6·334	4·500	5·417	3·5	12·1	229·41	
	6·334	4·500	5·417	1·8	12·1		117·982
U/S Glacis	4·5	3·0	3·75	1·5	10·0	56·25	
	3·555	1·69	2·623	1·5	12·1		47·607
D/S Glacis	5·295	1·583	3·439	5·5	12·1	228·865	
D/S Cut off	4·53	3·325	3·928	1·5	12·1	71·284	
	4·53	0	2·265	3	12·1		82·22
Pier	4·5	0	2·25	4·5	2·1	21·263	
Gate	3·0	0	1·5	3·0	10·0	45·0	
					Total	652·072	247·809
					Net	−247·809 =404·263	
					Say	404 Tonnes	

(7) Hydro dynamic Force

Pond level $=21\cdot 0$

U/S cut off level $=13\cdot 0$

$P_e=C\lambda\, w\lambda \quad =0\cdot 73\times 0\cdot 1\times 1\times 8=0\cdot 584$

$V_e=0\cdot 726\times P_e\times y=0\cdot 726\times 0\cdot 584\times 8=3\cdot 39$ tonnes/m

$V_e=$ per bay $=3\cdot 39\times 12\cdot 1=41$ tonnes

No.	Force	Vertical ↓ Tonnes	Vertical ↑ Tonnes	Horizontal →tonnes
1.	Raft & friction blocks	3058		
2.	Pier	1320		
3.	Wt. of water	649		
4.	Uplift		2580	
5.	Water Pressure			404
8.	Road Bridge	135		
9.	Gate Bridge	31		
	Total	5193	2580	404
	(*i*) Net (non E/Q condn.)		2613	404
6.	Boxed Sand		607	
	(*ii*) Net (non E/Q with boxed sand)		3220	404
7.	Hydrodynamic force			41
8.	Inertia force ($\alpha \times 5151$)		258	515
	or ($\alpha \times 4544$)		227	454
	(*iii*) Net (E/Q without boxed sand)		2355	960
	(*iv*) Net (E/Q with boxed sand)		2962	960

(*i*) Sliding factor (Non E/Q) $= \dfrac{C+N \tan\phi}{H}$

$$= \frac{0{\cdot}35 \times 56 \times 12{\cdot}1 + 2613 \times {\cdot}384}{404}$$

$$= 3{\cdot}07$$

(*ii*) Sliding Factor (Non E/Q)

$$= \frac{237{\cdot}16 + 3220 \times {\cdot}384}{404}$$

$$= 3{\cdot}65$$

(*iii*) Sliding Factor (E/Q)

$$= \frac{237{\cdot}16 + 2{\cdot}386 \times {\cdot}384}{960} = 1.21$$

(*iv*) Sliding Factor (E/Q)

$$= \frac{237{\cdot}16 + 2962 \times {\cdot}384}{960}$$

$$= 1{\cdot}432$$

APPENDIX 7.6

Typical Design of Pier

The Barrage has 9 single piers of 2·1 m width and 2 double piers of 4·2 m width. The piers are made monolithic with the raft and the reinforcement bars of the piers are well anchored into the raft. The piers are divided into different zones from upstream end to the downstream end for analysis due to different intensities of loading. For the design the worst combination of the following loads and moments have been considered.

(*a*) Dead loads

(*b*) Live loads due to road/railway bridge.

(*c*) Impact and Braking effects of live loads.

(*d*) Temperature forces transmitted through bridge bearings.

(*e*) Gate stoplog and the Hoist Bridge dead and live loads.

(*f*) Braking effect of Gantry crane.

(*g*) Uplift.

(*h*) Differential hydrostatic pressure with one side gate open and the other adjacent gate closed.

(*i*) Seismic forces and moments if any.

(*j*) Hydrodynamic forces due to Seismic conditions if any.

The reinforcement of the piers have been curtailed at convenient levels as per requirements. Typical calculations for zone No. 3-3 of under sluice pier where the effect of gate bridge, road bridge and differential head also would be felt are given below.

Zone 3-3 at R.L. 513·5

This zone has the effect of both gate as well as road ridge in addition to the safe wt. of pier.

Gate Bridge

Dead and live load Reaction$=36{\cdot}7$

Braking force$=0{\cdot}2\times 12=2{\cdot}47$

Dispersion width at R L. 513·5

$$=4+(523-516{\cdot}5)+(521{\cdot}5-513{\cdot}5)$$

$$=4+6{\cdot}5+8=18{\cdot}5 \text{ m}$$

N.E.Q.

$$V=\frac{36}{18{\cdot}5}=1{\cdot}95\ T$$

$$H=\frac{2{\cdot}4}{18{\cdot}5}=0{\cdot}13$$

$$M=0{\cdot}13\times 9{\cdot}5=1{\cdot}235 \text{ Tm.}$$

E.Q. (1+αv)

$$V=1{\cdot}05\times 1{\cdot}95=2{\cdot}05$$

$$H=1{\cdot}05\times 0{\cdot}13+0{\cdot}1\times 1{\cdot}95$$

$$=0{\cdot}136+0{\cdot}195=0{\cdot}33$$

$$M=0{\cdot}33\times 9{\cdot}5=3{\cdot}13 \text{ Tm}$$

E.Q. (1−αv)

$$V=0{\cdot}95\times 1{\cdot}95=1{\cdot}85 \text{ T}$$

$$H=0{\cdot}33 \text{ T.}$$

$$M=3{\cdot}13 \text{ Tm.}$$

Road Bridge

$\Sigma V=179{\cdot}89$ T is calculated for spill-way pier

$\Sigma H=7.83$ T

Dispersion width at R.L. 513·5

$$=10+(519{\cdot}5-513{\cdot}5)+(521{\cdot}5-516{\cdot}5)$$

$$=10+6+5=21 \text{ m}$$

N.E.Q.

$$V=\frac{179{\cdot}89}{21}=8{\cdot}2 \text{ T/m}$$

$$H=\frac{7{\cdot}83}{23}=0{\cdot}465 \text{ T/m}$$

Moment

$$0{\cdot}465\times 9{\cdot}9=4{\cdot}42 \text{ Tm}$$

E.Q. (1+αv)

$$V=1{\cdot}05\times 8{\cdot}2=8{\cdot}6$$

$$H=1{\cdot}05\times 0{\cdot}465+0{\cdot}1\times 8{\cdot}2$$

$$=0{\cdot}49+0.82=1{\cdot}31$$

$$M=1{\cdot}31\times 9{\cdot}5=12{\cdot}5 \text{ Tm.}$$

E.Q. (1−αv)

$$V=0{\cdot}95\times 8{\cdot}2=7{\cdot}8 \text{ T}$$

$$H=1{\cdot}31$$

$$M=12{\cdot}5 \text{ Tm}$$

Self Wt. of pier in zone 3-3 at R.L. 513·5

Wt. of pier between R.L (523·0−521·5)

$$=2{\cdot}1\times 6{\cdot}5\times 2{\cdot}4\times 1{\cdot}5=49 \text{ T}$$

Dispersion Width $=6{\cdot}5+(521{\cdot}0-516{\cdot}5)$

$$(+521{\cdot}5-513{\cdot}5)$$

$$=6{\cdot}5\times 4{\cdot}5+8=19 \text{ m}$$

$$\text{Wt./m}=\frac{49}{19}=2{\cdot}58 \text{ T/m} \qquad \ldots(1)$$

Between R.L.(521·5−519·5)

Wt. of pier $=2{\cdot}1\times 22\times 2{\cdot}4\times 2=222$ T.

Dispersion width $=22+(519{\cdot}5-516{\cdot}5)$

$$+(519{\cdot}5-513{\cdot}5)$$

$$=22+3+6=31 \text{ m}$$

$$\text{Wt/m}=\frac{222}{31}=7{\cdot}17 \text{ T/m} \qquad \ldots(2)$$

Between R.L. (519·5−517·5)

$$=2{\cdot}1\times 39\times 2{\cdot}4\times 2=394 \text{ T}$$

Dispersion Width $=39+4=43$ m

$$\text{Wt/m}=\frac{394}{43}=9{\cdot}15 \text{ T/m} \qquad \ldots(3)$$

Betweet R.L. (517·5−513·5)

$$\text{Wt/m}=2{\cdot}1(517{\cdot}5-513{\cdot}5)\times 2{\cdot}4$$

$$=2{\cdot}1\times 4\times 2{\cdot}4=20{\cdot}2 \text{ T/m} \qquad \ldots(4)$$

Total wt. of pier/m

$$=(2{\cdot}58+7{\cdot}17+9{\cdot}15+20{\cdot}20)$$

$$=39{\cdot}10 \text{ T.}$$

N.E.Q.

$$V=39{\cdot}10 \text{ T}$$

$$H=0$$

$$M=0$$

E.Q. (1+αv)

$$V=1{\cdot}05\times 39{\cdot}10=40{\cdot}08 \text{ T.}$$

$$H=0{\cdot}1\times 39{\cdot}10=3{\cdot}91 \text{ T}$$

$$M=0{\cdot}1\{2{\cdot}58\times 8{\cdot}75+7{\cdot}17\times 7+9{\cdot}15\times 5+20{\cdot}2\times 2\}$$

$$=0{\cdot}1\{22{\cdot}6+50{\cdot}19+45{\cdot}75+40.4\}$$

$$=0{\cdot}1\times 158{\cdot}94=15{\cdot}89 \text{ Tm}$$

E.Q. (1−αv)

$$V=0{\cdot}95\times 39{\cdot}1=37{\cdot}0$$

$$H=3{\cdot}91 \text{ T}$$

$$M=15{\cdot}89 \text{ Tm.}$$

Differential Head Moment

$$(d_2-d_1)=6{\cdot}60-1{\cdot}53=5{\cdot}07 \text{ m.}$$

Condition. Passing max. discharge taking d_1 on side of the pier and d_2 on the other side we get Differential Head moment.

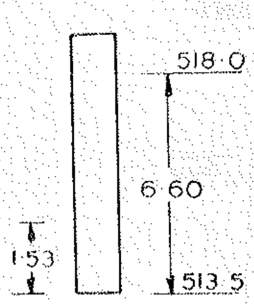


FIGURE 7.13

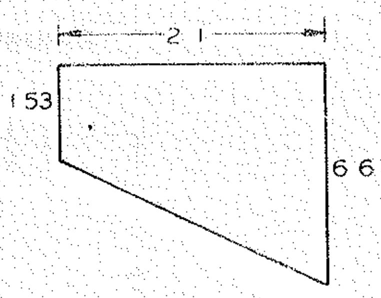


FIGURE 7.14

$$=\frac{1}{6}\left\{(6{\cdot}6)^3-(1{\cdot}53)^3\right\}$$

$$=\frac{1}{6}\left\{286-3{\cdot}6\right\}=\frac{282{\cdot}4}{6}$$

$$=47{\cdot}1 \text{ Tm}$$

Uplift Pressure

$$=1{\cdot}53\times2{\cdot}1+\tfrac{1}{2}\times5{\cdot}07\times2{\cdot}1=3{\cdot}21 \text{ t}$$

$$=\frac{5{\cdot}32}{8{\cdot}53 \text{ T}}.$$

Hydrodynamic Moments

$$p=h\times h\times H$$

$$=0{\cdot}875\times0{\cdot}1\times6{\cdot}6=0{\cdot}577$$

$$p=\frac{2}{3}\times0{\cdot}577\times6{\cdot}6=2{\cdot}54$$

$$M=2{\cdot}54\times\frac{2}{5}\times6{\cdot}6=6{\cdot}7 \text{ T m.}$$

	N.E.Q.		E.Q. $(1-\alpha k)$		E.Q. $(1+\alpha k)$	
	V	M	V	M	V	M
Road Bridge	8·2	4·42	7·8	12·5	8·6	12·5
Gate Bridge	1·95	1·24	1·85	3·13	2·05	3·13
Pier self wt.	39·1	—	37·0	15·89	40·08	15·89
Up lift	(−) 8·53	—	(−) 8·53	—	(−) 8·53	—
Diff. Head Moment	—	47·1	—	47·1	—	47·1
Hydrodynamic Moment	—	—	—	6·7	—	6·7
Net	40·72	52·76	38·12	85·32	42·92	85·32
	$e=\frac{52{\cdot}76}{40{\cdot}72}$	$=129{\cdot}5$ cm	$e=\frac{85{\cdot}32}{38{\cdot}12}$	$=225$ cm	$e=\frac{85{\cdot}32}{42{\cdot}92}$	$=199$ cm

CALCULATION OF REINFORCEMENT

Derivation of General formula :

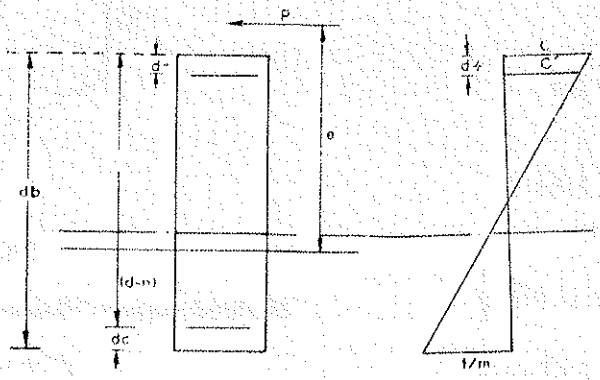


FIGURE 7.15

The pier has been designed as a column section 1 m×2 1 m subjected to combined bending and direct thrust equations used for calculations of reinforcement are given below :

Permissible Stresses (Earthquake condition)

$t=2100$ kg/cm²

$C=75$ kg/cm²

$M=15$

Notations

P=Direct load

e=Eccentricity of load

A=Area of Steel in cm² in compression

C'=Compressive stress in concrete at the level of compression steel.

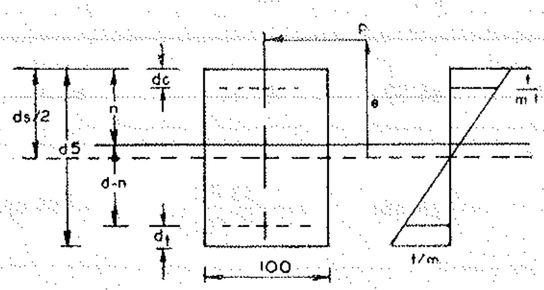


FIGURE 7.16

t=Tensile Stress in tension zone.

$$\frac{mc}{t}=\frac{n}{d-n}$$

$$t=\frac{mc\,(d-n)}{n}$$

$$=15\,\frac{c}{n}\,(200-n)$$

$$T=\frac{15c}{n}\,(200-n)\,At.$$

$$ds=210$$

$$d=210-10=200$$

$$dc=dt=10$$

$$Ac=At=A$$

$$C=\tfrac{1}{2}\,cbn=\tfrac{1}{2}\,c\times100\times n=50\,nc$$

$$=\frac{c}{n}\,(50^2\,n)$$

$$\frac{C}{n}=\frac{C'/m-1}{n-dc}$$

C'=Compressive stress in steel.

$$nC'=C\,(m-1)\;\frac{(n-dC)}{n}$$

$$C'=C'\ AC=C\ (m-1)\frac{(n-dC)}{n}AC$$

$$=C\times 14\ \frac{(n-10)}{n}AC$$

$$=14C\frac{(n-10)}{n}AC$$

$$=\frac{C}{n}\ (14\ ACn-140\ AC)$$

$$C+C'=\frac{C}{n}\ \{50\ n^2+14\ ACn-140\ AC\} \quad \ldots(1)$$

$$T=\frac{C}{n}\{3000\ At-15A\ tn\} \quad \ldots(2)$$

$$P=CA+C'AC-t_A t$$

$\therefore$ equating P=Total Stress.

$$P=CA+C'AC-t_A t$$

$$=\frac{C}{n}\ \{50\ n^2+14\ ACn-14\ CAC\}$$

$$-\frac{C}{n}\ \{3000\ At-15\ Atn\}$$

$$=\frac{C}{n}\{50\ n^2+14\ ACn+15\ Atn-140\ AC-3000\ At\}$$

$$P=\frac{C}{n}\ \{50\ n^2+29\ An-3140\ A\} \quad \ldots(3)$$

Taking moments about tensile steel.

$$P\ (e+95)=\frac{1}{2}\ Cbn\left(d-\frac{n}{3}\right)+C'AC\ (d-10)$$

$$P\ (e+95)=\frac{C}{n}\ (50\ n^2)\left(200-\frac{n}{3}\right)$$

$$+\frac{C}{n}\ (14\ An-140\ A)\ 190$$

$$=\frac{C}{n}\ 1667\ n^2\ (600-n)$$

$$+\frac{C}{n}\ \{2660\ An-26600\ A\}$$

$$=\frac{C}{n}\ \{10000\ n^2-1667\ n^3+2660\ An$$
$$-26600\ A\}$$

$$=\frac{1667\ C}{n}\ \{600\ n^2-n^3+160\ An-1600\ A\} \quad \ldots(4)$$

Dividing (4) by (3)

$$\left(\frac{e+95}{1}\right)=\frac{1667\ (600\ n^2-n^3+160\ An-1600\ A)}{50\ n^2+29\ An-3140\ A}$$

$$(e+95)\ (50\ n^2+29\ An-3140\ A)=1667\ (600\ n^2-n^3$$
$$+160\ An-1600)$$

$$n^3+n^2\ \{3\ (e+95)-600\}+An\ \{174\ (e+95)-160\}$$
$$=A\ \{189\ (e+95)-1600\} \quad \ldots(5)$$

Value of n' can be found out from.

$$t=\frac{150}{n}\ (200-n)\ \text{ and }\ C=\frac{Pn}{50n^2+29\ An-3140A}$$

$$NC=\frac{P}{50\ n+29\ A-3140\ \frac{A}{n}}$$

Zone 3—3 at R.L. 513·5

$e=225$ cm $\qquad P=38{\cdot}15$

$A=10{\cdot}6$ cm² *i.e.* 18 mm ϕ @240 c/c

$$n^3+n^2\{3(e+95)-600\}+An\{1{\cdot}74(e+95)-160\}$$
$$=A\{189(e+95)-1600\}$$

$$n^3+360n^2+10{\cdot}6n\times 396$$
$$=10{\cdot}6\{58900\}$$

$$n^3+360n^2+4200n=623000$$

$$C=\frac{38150}{50\times 35+29\times 10{\cdot}6-3140\ \frac{10{\cdot}6}{35}}$$

$$=\frac{38150}{1750+307-950}=\frac{38150}{1107}$$

$$=33{\cdot}7\ \text{kg/cm}^2$$

$$t=\frac{15C}{n}(200-n)$$

$$=\frac{15\times 33{\cdot}7}{35}\times 165$$

$$=2370\ \text{kg/cm}^2$$

More than 1·5×1400 in E.Q. condition let us provide 18 mm ϕ @150 c/c

$$A=16{\cdot}97\ \text{cm}^2$$

$$n^3+360n^2+17\times 396\times n$$

$$=17(58900)$$

$$n^3+360n^2+6725n=1000.000 \qquad n=42{\cdot}00n$$

$$C=\frac{34720}{50\times 42+29\times 17-31\,40\dfrac{17}{42}}$$

$$=\frac{37420}{2100+493-1270}=\frac{374}{132}$$

$$=28{\cdot}3\ \text{kg/cm}^2$$

$$t=\frac{15\times 28{\cdot}3}{42}(200-42)$$

$$=\frac{15\times 28{\cdot}3}{42}\times 158=1595\ \text{kg/cm}^2$$

Less than 1·5×1400=2100

Hence O.K.

$A_L=16{\cdot}97$ 18 mmϕ @ 150 centre

Typical Design of Abutment

The left and right abutment of the Barrage are 66·5 m long each. Depending on the intensity of loading over the entire length of the abutment, it has been divided into four blocks, namely (*a*) u/s block (*b*) Gate block (*c*) Road bridge block and (*d*) d/s block. The abutment has been designed as a retaining wall separated from the undersluice raft by an expansion joint with seals. Each block has been designed for the loads and moments acting on the block under consideration.

The top width of the abutment in each block has been fixed as per the requirements due to loads and moments, minimum width required for blockouts in the main gate and stoplog grooves, bridg bearing, hoist trestle foundation, etc.

For the design of the abutment, worst combination of the following loads and moments pertaining to the block under consideration has been taken into accounts

(*a*) Dead load

(*b*) Live load due to bridges

(*c*) Impact and braking effect of live loads

(*d*) Temperature force transmitted through bridge bearing

(*e*) Dead and live loads of gate and gate bridge

(*f*) Braking effect of gantry crane

(*g*) Earth pressure, live load surcharge and saturation pressure

(*h*) Uplift

(*i*) Seismic forces and moments if any

(*j*) Hydrodynamic forces due to seismic condition if any.

The abutment has been checked for safty against allowable bearing pressure, overturning and sliding. Typical calculations for the u/s abutment block are given below.

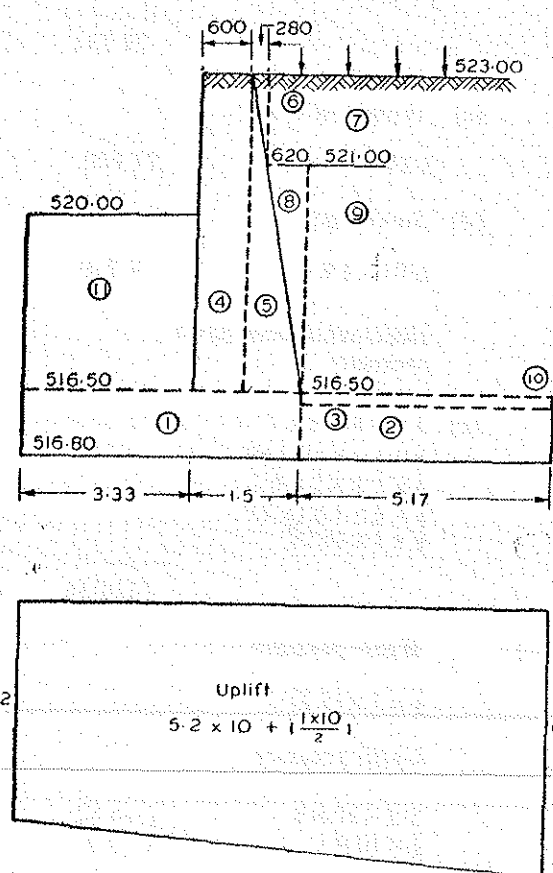


FIGURE 7.17

Non Earthquake Case

Sl. No.	Item	Vertical forces	Horizontal forces	Lever arm	Moment +ve	Moment −ve
A	(*a*) *Weight of Concrete*					
1.	4·83×2·4×1·7	19·700		2·42	47·60	
2.	5·17×1·00×2·4	12·400		7·40	91·65	
3.	½×5·17×0·7×2·4	7 960		6·55	52·14	
4.	·6×6·5×2·4	9·360		3·63	33·93	
5.	½×·9×6·5×2·4	7·020		4·23	29·6	
		56·440				
	(*b*) *Back Fill*					
6.	½×·28×2×1·8	0·504		4·12	2·075	
7.	5·79×2×1·8	20·820		7·10	148·00	
8.	½×4·5×0·62×2	2·790		4·52	12·60	
9.	5·17×4·5×2	46·500		7·42	345·00	
10.	½×5·17×0·7×2	3·620		8·28	30·00	
		74·234				
	(*c*) *Weight of Water*					
11.	3·33×3·5×1	11·650		1·67	19·45	
	(*d*) *Surcharge*					
	6·07×1·6	9·710		6·94	67·40	
B.	*Horizontal force earth pressure*					
	(*a*) 1·6×8·2×⅓		4·375	4·10		17·92
	½×⅓×1·8×(2)²		1·200	6·87		8·22
	⅓×1·8×2×6·2		7·435	3·10		23·25
	½×⅓×1×(6·2)²		6·400	2·07		13·25
	½×1×(6·2)²		19·200	2·07		39·70
		152·034	38·610			
(−)	*Water pressure*					
	½×1×(5·2)²		−13.50	1·73	23·30	
C	*Uplift pressure*					
	5·2×10×1	(−) 52		5·00		260·00
	½×10×1	(−) 5		6·67		33·00
	Total	95·034	25·110		902·745	395·34

$$n = \frac{902{\cdot}745 - 395{\cdot}34}{95{\cdot}034}$$

$$= \frac{507{\cdot}405}{95{\cdot}034} = 5{\cdot}34$$

Eccentricity $= 5{\cdot}34 - 5 = 0{\cdot}34$ m.

$e = 34$ cm

Normal pressure

$$= \frac{Ev}{b}\left(1 \pm \frac{6e}{b}\right)$$

$$P_r = \frac{95{\cdot}034}{10}\left(1 + \frac{6 \times {\cdot}34}{10}\right)$$

$$= \frac{95{\cdot}034}{10}\ (1 \pm {\cdot}204)$$

$= 11{\cdot}44$ and $7{\cdot}57$

Factor of safty for sliding

$$\frac{\Sigma V}{\Sigma H}\ \tan\phi$$

$$= \frac{95{\cdot}034}{25{\cdot}11}\ \tan 30°$$

$$= \frac{95{\cdot}034 \times {\cdot}5765}{25{\cdot}11}$$

$= 2{\cdot}18$

Earthquake Case

(*i*) $(1 - \alpha V)$ condition

Sl. No.		Item	Vertical force	Horizontal force	Lever arm	Moment (+) ve	Moment (—) ve
		Total NEQ forces	95·034	25·110		902·745	395·34
		Deduction in moments due to αV (0·5)	(—) 7·12				
A	(*a*)	*Weight of Concrete*					
1.		$4{\cdot}83 \times 2{\cdot}4 \times 1{\cdot}7$				2·380	
2.		$5{\cdot}17 \times 1{\cdot}80 \times 2{\cdot}4$				4·5825	
3.		$\frac{1}{2} \times 5{\cdot}17 \times 0{\cdot}7 \times 2{\cdot}4$				2·61	
4.		$6 \times 6{\cdot}5 \times 2{\cdot}4$				1·7	
5.		$\frac{1}{2} \times {\cdot}9 \times 6{\cdot}5 \times 2{\cdot}4$				1·48	
	(*b*)	*Back fill*					
6.		$\frac{1}{2} \times {\cdot}28 \times 2 \times 1{\cdot}8$				0·10375	
7.		$5{\cdot}79 \times 2 \times 1{\cdot}8$				7·400	
8.		$\frac{1}{2} \times 4{\cdot}5 \times 0{\cdot}62 \times 2$				0·630	
9.		$5{\cdot}17 \times 4{\cdot}5 \times 2$				17·250	
10.		$\frac{1}{2} \times 5{\cdot}17 \times 0{\cdot}7 \times 2$				1·500	
						39·636	

Earthquake Case *(Contd.)*

Sl. No.		Item	Vertical force	Horizontal force	Lever arm	Moment (+) ve	Moment (−) ve
	(c)	*Surcharge*					
		6·07×1·6				3·370	
		Horizontal Force *Earth pressure*					
		1·6×8·2×·06		0·788	4·375		3·45
		½×·06×1·8×2^2		0·21	7·53		1·5
		0·06×1·8×2×6·2		1·34	3·1		4·15
		½×0·226×1×6·2		4·35	4·1		17·8
				6·888			26·90
		Inertia Forces					
A	(a)	*Weight of Concrete*					
1.		4·83×2·4×1·7×·1		1·97	0·85		1·68
2.		5·17×1·00×2·4×0·1		1·24	0·5		0·62
3.		½×5·17×0·7×2·4×0·1		0·79	1·23		0·97
4.		·6×6·5×2·4×0·1		0·936	4·95		4·65
5.		½×0·9×6·5×2·4×0·1		0·702	3·867		2·7
	(b)	*Back fill*					
6		½×·28×2×1·8×·1		0·0504	6·87		0·343
7.		5·79×2×1·8×·1		2·082	7·2		15·00
8.		½×4·5×0·62×2×·1		0·279	4·7		1·08
9.		5·17×4·5×2×·1		4·65	3·95		18·40
10.		½×5·17×0·7×2×·1		0·3620	1·47		0·532
							35·355
		Surcharge					
		6·07×1·6×0·1		0·971	8·2		7·95
		Total E.Q. Moment	87.914	45·83		944·566	477·115

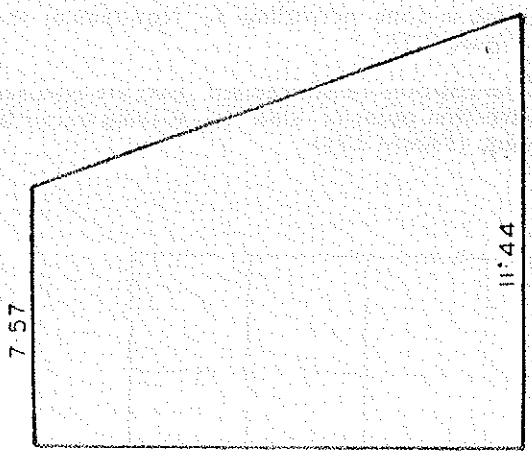


FIGURE : 7.18

$$\bar{x} = \frac{944{\cdot}566 - 477{\cdot}115}{87{\cdot}914}$$

$$= \frac{467{\cdot}451}{87{\cdot}914} = 5{\cdot}317$$

$$= 5{\cdot}32$$

Eccentricity $e = 5{\cdot}32 - 5 = 0{\cdot}32$

Normal pressure

$$P_r = \frac{87{\cdot}914}{10}\left(1 \pm \frac{6 \times {\cdot}32}{10}\right)$$

$$= \frac{87{\cdot}94}{10}\ (1 \pm {\cdot}192)$$

$$= 10{\cdot}48 \text{ and } 7{\cdot}10 \text{ kg/m}^2$$

Factor of safety for sliding

$$= \frac{\Sigma V}{\Sigma H} \tan \phi$$

$$= \frac{87{\cdot}914}{45{\cdot}83} \times {\cdot}5765$$

$$= 1{\cdot}105$$

$$= 1{\cdot}11$$

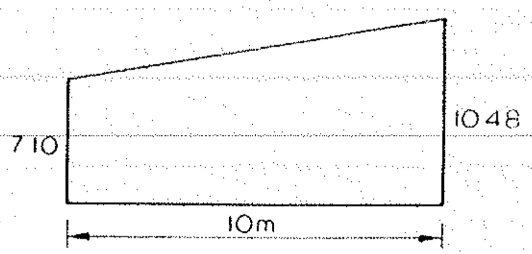


FIGURE 7.19

Calculating the moments at various levels as before, max. moment at the bottom of stem

$$= 85400 \text{ M kg}$$

$$At \text{ required} = \frac{85400}{0{\cdot}87 \times 140 \times 1400 \times 1{\cdot}5}$$

$$= 33{\cdot}4 \text{ cm}^2$$

provide 20 mm ϕ bar @ 150 mm c/c in two rows.

At base curtailment at different levels has been done as per requirement shown in the diagram.

Design of Base Slab

The pressure diagram for different conditions *i.e.*, non earthquake and earthquake conditions are worked out and the governing diagram is taken for moment calculations.

The Max. moment in toe portion due to dead weights uplift and soil pressure is calculated to be about 52710 M kg.

$$At \text{ required} = \frac{52710 \times 100}{0{\cdot}87 \times 160 \times 1{\cdot}5 \times 1400}$$

$$= 18 \text{ cm}^2.$$

provide 20 mm ϕ bar at 150 mm c/c.

Similarly, the max. moment in head portion due to dead weights, uplift and soil pressure is calculated to be about 62400 M kg.

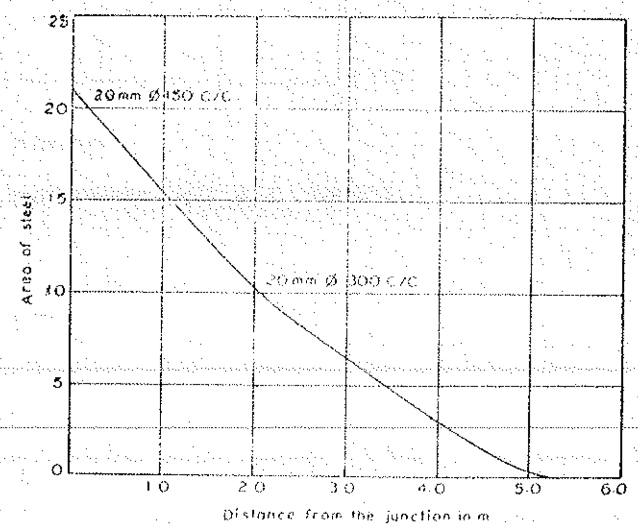


FIGURE : 7.20 U/S Abutment Block : Heel Reinforcement Curtail Diagram.

$$At \text{ required} = \frac{62400 \times 100}{0.87 \times 160 \times 1.5 \times 1400}$$

$$= 21.3 \text{ cm}^2.$$

provide 20 mm ϕ bar @ 150 mm c/c.

Curtailment of the rods at different lengths of the heel slab is done according to requirement shown in the Figure 7.20.

The required reinforcement bars are adjusted from construction point of view by varying dia of bar and spacing. (Figure 7.21)

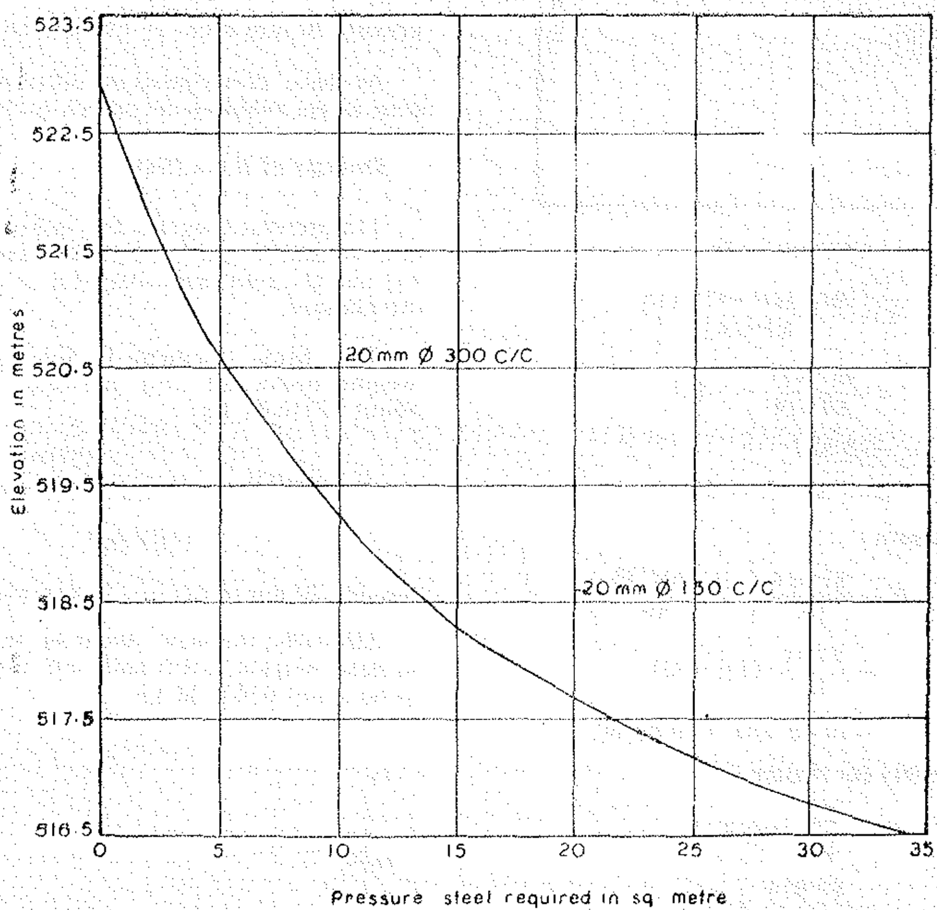


FIGURE : 7.21 U/S Abutment Block ; Stem Reinforcement Curtailment diagrams.