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Manual on Barrages and Weirs on Permeable Foundation

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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 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 Foundation 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.

CVJ 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 head-works 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

Silt Exclusion

1.0 Introduction

Rivers flowing in erodible valleys carry heavy sediment load, particularly in the rainy season. In the river, the heavy detritus rolls along the bed, the coarser particles are concentrated in lower layers and the comparatively lighter once are suspended in upper layers. The high sediment charge if passed from the river into the off-taking artificial channel, whether irrigation, power or navigation, is always a source of great trouble. It is, therefore, essential to provide measures for controlling the sediment in the off-taking channel.

1.0.1 It is never possible, and usually never necessary to eliminate the sediment completely from a channel. The sediment load permitted in a channel should be restricted so as to be within its sediment carrying capacity. Even if the load is within the transporting capacity it may sometimes become necessary to remove portion of it depending upon the specific purpose for which the channel is to be used. Thus in the case of irrigation channels, coarse sand particles should be eliminated as these are detrimental to the fields. On the other hand, fine silt should be allowed, as this adds to the fertility of the fields, helps in preventing weed growth in the channel and reduces seepage losses. In the case of hydropower channels, it may be necessary to remove even fairly fine particles as these cause damage to the turbine bearings.

1.0.2 The various devices used for controlling sediment are classified as preventive and curative. Preventive devices are those which help in preventing entry of the sediment into the channel and are located on the upstream of the head regulator. Curative devices are those which are used for extracting from the channel some portion of the sediment which has already entered it and are provided at some convenient site in the channel itself. The devices mainly used are sediment or silt excluders, sediment or silt extractors or ejectors, sand and gravel traps and settling basins. Of these, the first one is preventive and others curative.

1.0.3 *Sediment Extractor* : Sediment extractor (or ejector) is the device sited across the canal bed downstream of the head regulator and is intended for removing a portion of bed sediment which has already entered into the canal. It is a measure and primarily meant for extracting the sediment moving along the bed by extracting and ejecting it. If necessary and provided suitable sites exist, more than one extractor can be made for a channel. In case it is intended to remove certain portion of the suspended sediments also, it will be necessary to provide some sort of settling basin or stilling pool upstream of the extractor.

1.0.4 In this Chapter, sediment/silt excluders of the preventive type only are discussed. Silt excluder is a device located just in front of the canal head regulator and its function is to exclude the lower layers of river water which carry the comparatively coarses particles and prevent them from being diverted into the canal.

1.1 Basic Facts in Silt Exclusion

The silt exclusion takes advantage of the basic facts that :

(*i*) In a flowing stream, the concentration of silt charge as well as the size of silt in the lower layers is higher than that in the upper ones.

(*ii*) The silt gets agitated due to obstruction or sudden change in flow condition.

(*iii*) The higher the velocity, the more is the bed silt thrown up.

1.2 Requirements of Silt Exclusion

The requirements of satisfactory silt exclusion can thus be mentioned as :

(*i*) It should draw relatively silt-free water from the top layers and exclude heavily laden bottom layers ;

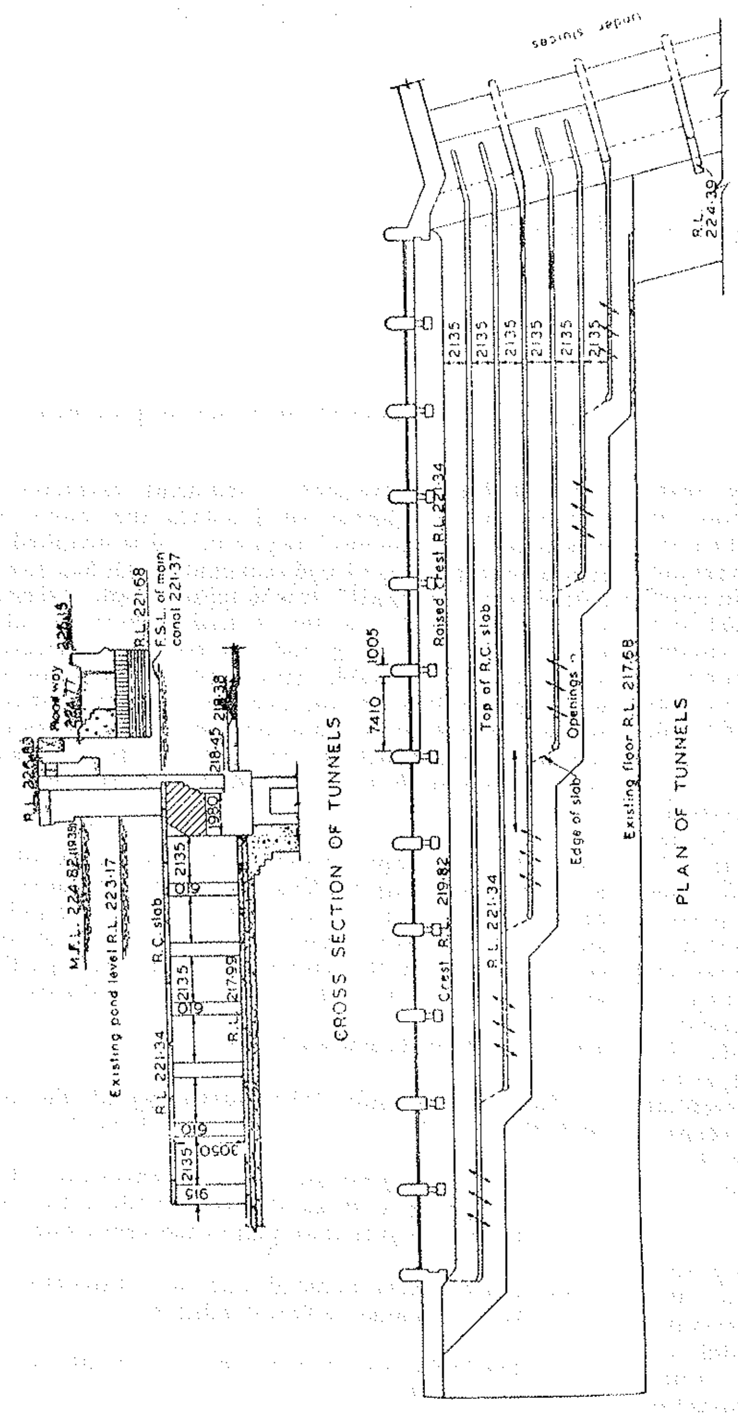


FIGURE 1.1 : General Layout of Khanki Type of Silt-Excluder.

(*ii*) Entry of water should be smooth so as to avoid turbulence in water and agitating of silt ;

(*iii*) It should provide a smooth surface by paving or by plastering the bed and the sides to reduce friction and give chance to the silt in the upper layers to settle down in the layers adjacent to the bed from where it is led away downstream ; and

(*iv*) Velocity should be reduced in the pocket so as to attract silt load in lower layers.

1.3 Various Methods of Silt Exclusion

For regulating the supplies into the canal with the silt excluded as much as possible, there are different methods including the use of special devices. These are mentioned below :

(*a*) The undersluices may be partially opened to release the surplus water over the crest or through silt exluders of undersluices so as to draw clearer water into the regulator. The velocity in the pocket is generally kept low (0.3 to 0.6 m/sec) so as to keep the silt laden water in lower layers. Crest level of the head regulator is kept higher than the crest level of undersluices so as to draw clearer water into the canal.

(*b*) In the second method, called the Still Pond Regulation, the scouring sluices are kept closed when the canal is running. Only as much supply enters the pocket as is required for the canal and the surplus is escaped over some other section of the diversion structure. The velocity of water in the pocket is, therefore, very much reduced as a smaller discharge enters through the same waterway. The silt is thus enabled to settle down and relatively clear water enters the canal. This method is only practicable where the crest level of the canal head regulator is sufficiently higher than that of the upstream floor of the undersluices. The silt is allowed to accumulate in the pocket till it reaches to within about 0.5 m of the crest. The canal is then closed and the sluices opened till the entire deposit is washed away. Till the scouring operations are over, the canal supply would be interrupted. The drawback is that the canal would have to be closed at periodic intervals for scouring operations, resulting in wastage of discharge and loss of irrigation to that extent.

(*c*) Special works may be constructed to control sediment entering into the canal. These include the preventive and curative devices. These are provided where these are considered necessary.

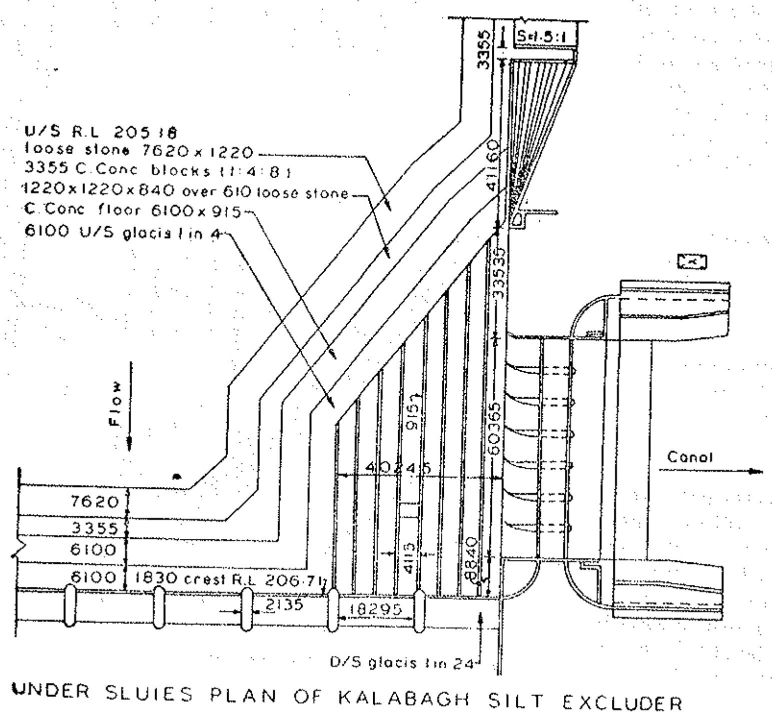
(*d*) The location of the headworks itself can be planned such that water is abstracted from the outside of a curved reach of the river since the river carries less sediment load at the outside of a curve than the average sediment load of the river and there is concentration of sediment on the inside of the bend. A curved divide wall to provide a curved approach towards the canal head regulator is also some times adopted.

1.4 Types of Silt Excluders

The silt excluders which are in vogue in India are classified into two types, viz., Khanki type and Kalabagh (or Trimmu) type. These two types have derived their names from the diversion structures where they have been provided.

1.4.1 In the Khanki type, the location of the upstream mouths of the tunnels is staggered so that the length of the tunnels decreases in a series of steps, the longest tunnel being nearest to the head regulator. In the Kalabagh type the mouths of the tunnels at the upstream end are in one straight line. For the general layout of the two original works, see Figures 1.1 and 1.2

1.4.2 The efficiency of any type depends on a complex combination of factors like approach conditions in the river, quality and quantity of the suspended and



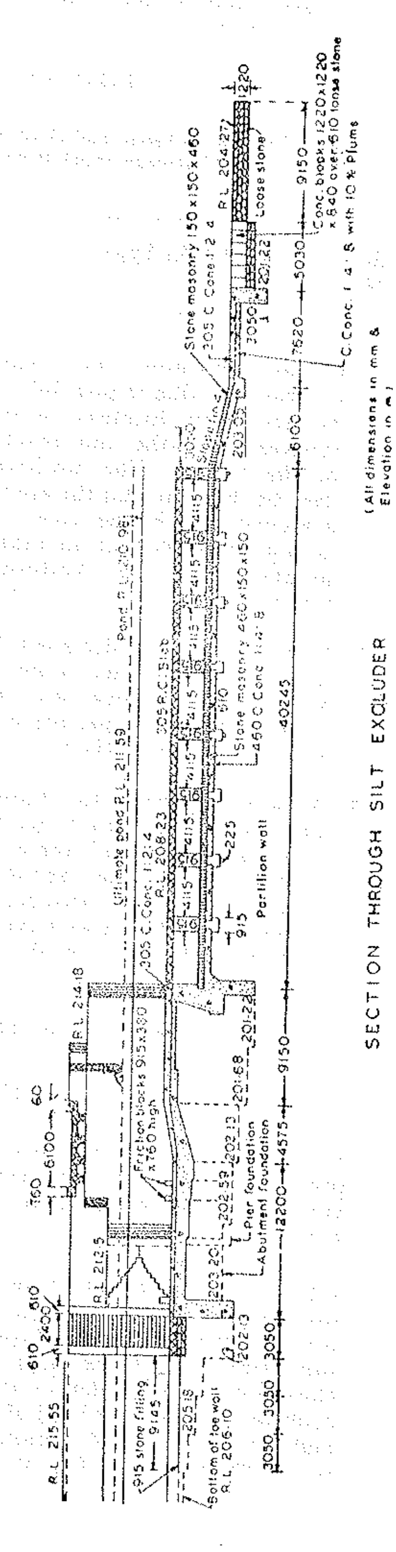


FIGURE 1.2 : General Layout of Kalabagh Type of Silt Excluder

bed sediment, discharge, etc One type which may be suitable for one set of conditions may be quite inefficient under another set of conditions. In general the comparative merits and demerits of the two types are given in Table 1.1.

TABLE 1.1

Comparative Merits and Demerits of Khanki & Kalabagh Types of Excluders

Sl. No.	Khanki Type	Kalabagh Type
1.	Greater flexibility in design is available due to possibilities of varying the location of the upstream mouths and lengths of the tunnel. The positions can be fixed to suit the approach conditions in the river. It is not necessary that the tunnel mouths may be uniformly spaced.	Degree of flexibility is less.
2.	If the tunnels are designed for a particular approach condition which changes later on, the adverse effect on performance is usually more marked.	This is not so sensitive to changes in the approach conditions.
3.	Mouths of some of the tunnels are located in front of the head regulator and there is more possibility of sediment thrown up in to suspension due to disturbance at entry in to tunnel, being passed into the canal.	All the mouths are located well upstream of the head regulator and the chances are comparatively less.
4.	More complicated to construct.	Simpler to construct.

1.4.3 Khanki type has been more commonly used, probably on account of its greater flexibility. Typical layout of a silt excluder in a barrage is shown in Figure 1.3.

1.4.4 In diversion structures across rivers with fairly steep slopes which bring in big sized rolling stones, it would be advisable to adopt silt deflectors with open top instead of silt excluder tunnels which may get choked up. These deflectors reduce the maintenance problems. The crest level of head regulator should be higher than the top of deflector by at least 0.5 to 1 m so as to prevent entry of coarse silt into the head regulator. Typical layout of a silt deflector in a barrage is shown in Figure 1.4.

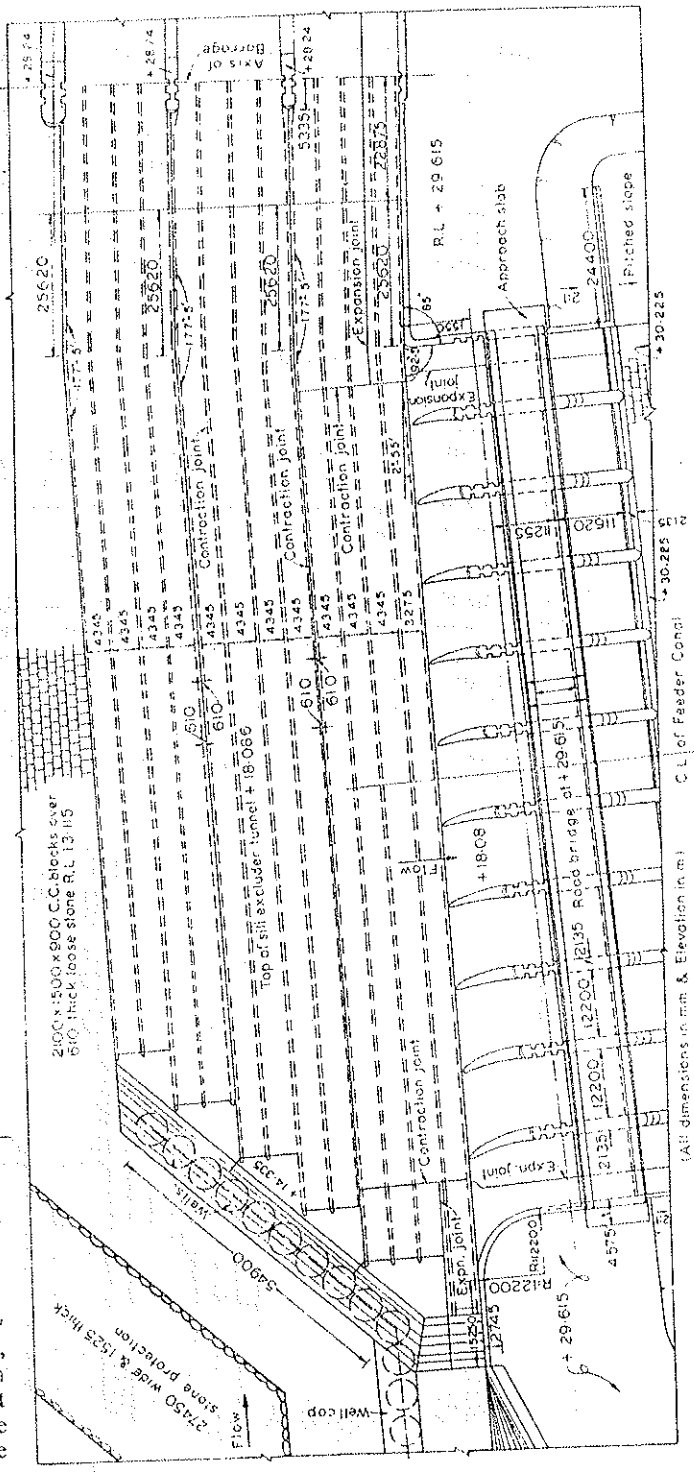


FIGURE 1.3 : Silt Excluder Bays in a Barrage Typical Layout.

1.5 Layout of Silt Excluder and Silt Deflector

The layout of a silt excluder consists of a series of parallel tunnels covered with a diaphragm slab, sited in one or more bays of the undersluices adjoining the head regulators and running parallel to their alignments.

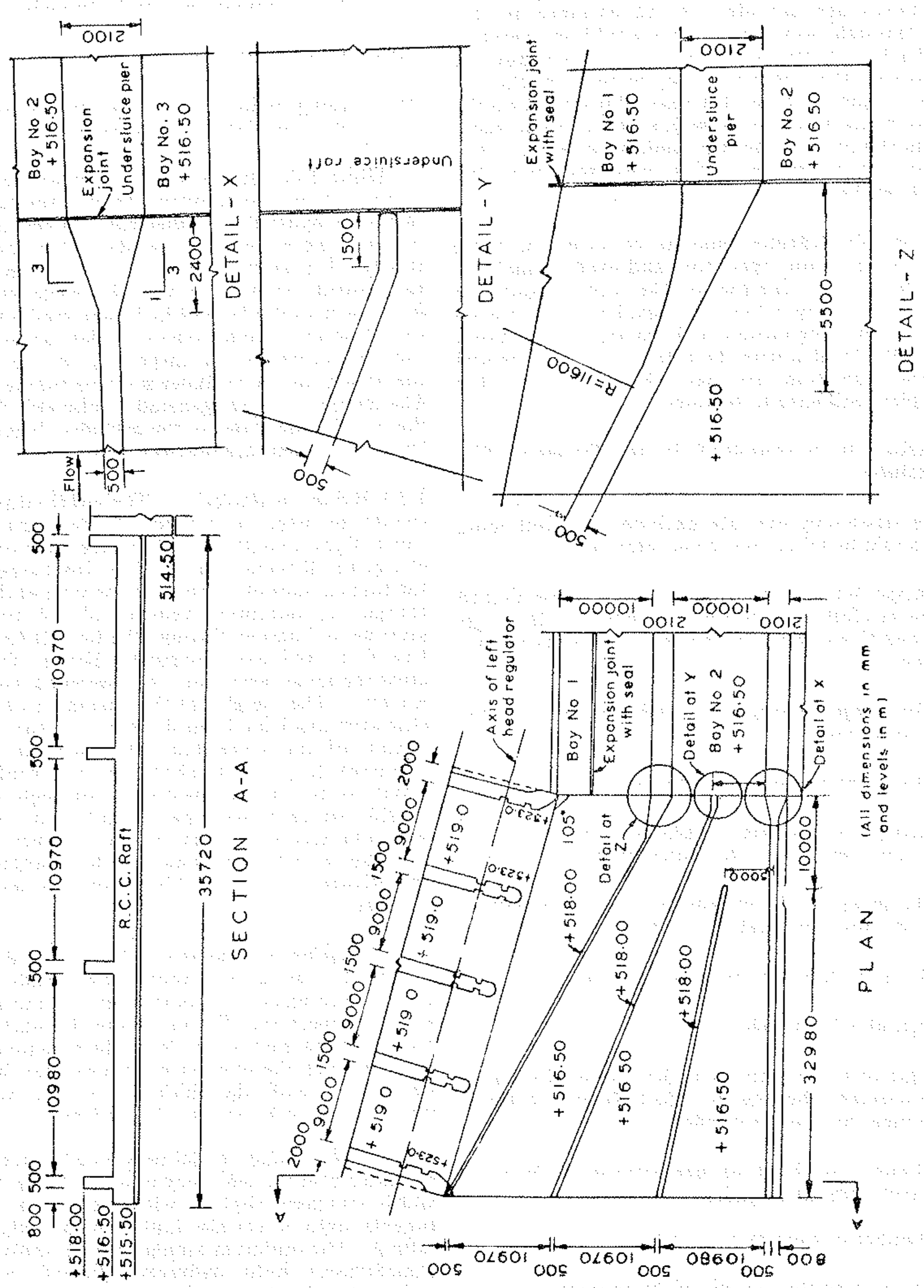


FIGURE 1.4: Typical Layout of Silt Deflector.

The downstream openings of the tunnels are located at the crest of the undersluice bays. The crests of the head regulators are kept either slightly higher or at the same level as the top of the diaphragm slab. For cleaning and repairing of the tunnels whenever necessary, grooves are provided at the entrance to the tunnels through which stop logs would be operated. The undersluice gates could be used to control the flow through the tunnels. The portion of the silt excluder tunnels beyond the undersluice floor on the upstream would be provided with seals at their junction with the portion over the undersluice floor. The tunnel walls would be monolithic with the base slab of the silt excluder.

1.5.1 The silt deflector consists of three or more deflector vanes with open top and fixed monolithic with the base slab. The top of the vanes would be kept at about 0 5 m below the crest level of the head regulator The alignments and spacings of the vanes are generally decided with the help of hydraulic model tests. The transition between the vanes and the barrage piers will have to be smooth.

1.6 Factors to be considered in the Design of Silt Excluder

While designing the silt excluder, the following factors should be taken into consideration :

(*i*) Approach conditions in the river. These should be carefully studied for a number of discharge conditions, both at site and by model experiments.

(*ii*) Discharge through undersluices for various conditions.

(*iii*) Discharge through head regulator.

(*iv*) Charge and grade of sediment in the river water coming into the undersluices.

(*v*) Charge and grade of sediment can be permitted to go into the canal.

(*vi*) Head available for operating the excluder.

1.7 Design of Silt Excluder

The type of silt excluder, viz., Khanki or Kalabagh has to be selected. Besides this, the following features of the excluder have to be decided :

(*i*) Number of bays of undersluices to be provided with silt excluder tunnels.

(*ii*) Number of tunnels in each bay.

(*iii*) Location of the mouths of the tunnels.

(*iv*) Velocity and discharge through the tunnels.

(*v*) Layout and shape of the excluder in plan.

(*vi*) Longitudinal sections of tunnels.

(*vii*) Cross sections of tunnels.

(*viii*) Longitudinal and side projections of diaphragm slab over a tunnel.

While full aspects of the designs are not covered by theoretical background, some aspects are determined by hydraulic calculations. Others are based on previous experience and quite a few are fixed arbitrarily. It is essential that in case of major works, the salient features of the silt excluder tunnels are finalised with the help of hydraulic model tests. Since the structure would remain under water most of the time and occasions for inspecting it are rather less, the design of the excluder must be simple and robust. The design criteria generally followed for fixing up the broad dimensions of the excluder before they are tested by models are outlined below.

1.7.1 *Width of Excluder* : The width of the excluder should be fixed in relation to the number of undersluice bays (length of undersluice portion) and discharge of off taking canal. The discharge carried by the tunnels should be about 20 percent of the canal discharge. A minimum velocity of 1.8 to 3.0 m/sec must be maintained through the tunnels to keep them free from sediment deposit. Hence the required cross-sectional area can be obtained as discharge/velocity. The height of the opening is determined by clearance available from the floor of the pocket upstream of the undersluices to the crest of the head regulator less the thickness of the roof covering. Total width of the waterway through the tunnels would be cross-sectional area/height. This width would be accommodated by the number of tunnels plus the wall thickness and has to be adjusted so that the excluder can be provided in one or more number of bays.

1.7.2 *Number of Tunnels* : While no definite rule exists, the size of the tunnel is generally fixed based on the convenience of construction, maintenance and ease of inspection. The minimum size normally adopted is about $2m \times 1.5m$. With the selection of the size of the tunnel, the number of tunnels per bay can be worked out with the thickness of the walls of the tunnel taken to be about 0.4 to 0 5 m.

1.7.2.1 Hydraulic model tests on silt excluders have indicated that the efficiency of an excluder of a given width increases slightly with increasing number of tunnels upto a certain limit beyond which it starts falling. The optimum number which must take into consideration both hydraulic efficiency and cost has to be worked out by trial and error.

1.7.2.2 Sometimes the silt excluder tunnels are arranged in two storeys as in the case of Nangal dam.

1.7.3 *Location of the Mouth of Tunnels*

The mouths of the tunnels should cover the entire zone of inflow to the head regulator. For deciding the location of the mouths, the model of the excluder should be tested for river approach as it would exist in final stable conditions. It is essential that the hydraulic model should be run for a sufficiently long time to reproduce the final conditions.

1.7.3.1 In the Kalabagh type, all the mouths lie in one straight line. The exact distance of this line upstream of the head regulator has to be fixed by model tests.

1.7.3.2 In the Khanki type, the mouths are spaced uniformly. In certain old design, the mouths were located at selected points based on approach conditions of the river. It was found that due to slight changes in approach, there was a sharp fall in efficiency. With uniformly spaced mouths, variations in approach, conditions affect the performance to a much lesser degree.

1.7.3.3 In certain old design, each tunnel was provided with one additional inlet in the side. This was found to decrease the efficiency. In recent practice, only one inlet for each tunnel at its upstream end is provided.

1.7.4 *Velocity and Discharge through Tunnels*

The velocity obtained in any tunnel should be such as to flush out all the sediment coming into it. It varies from about 1.8 to 3.0 m/sec. There should be adequate head available to produce this velocity under all operating conditions and stages of the river.

1.7.4.1 The tunnels normally run as pressure conduits. However, there could be situations when flow with free surface may take place near the tail end of the tunnels. For such conditions, it should be ensured that there is no possibility of a hydraulic jump forming inside the tunnel.

1.7.4.2 Each tunnel must take the same discharge as others inspite of their unequal lengths. This is achieved by suitably adjusting the cross section at intermediate points such that the total loss of head in each is the same.

1.7.4.3 The diaphragm slab should separate the upper and lower layer of waters with minimum turbulance. This would be possible if the entire discharge below the level of the diaphragm is taken into the tunnel.

1.7.4.4 In order to prevent sudden changes in velocity at the diaphragm slab, the tunnel entrances are bell-mouthed both in plan and vertical section. The widths provided at the entrance are gradually reduced to the tunnel dimensions through suitable transitions.

1.7.5 *Layout and Shape of Excluder in Plan*

The layout and shape are designed so as to command entire zone of inflow towards the head regulator. The layout has to fit in with the alignments of the undersluices and the head regulator. All connecting bends should be curves of large radii. All the tunnels are now-a-days provided with identical widths at the entrance and at exit. Intermediate sections are adjusted in plan. At the upstream end, bell mouthing is provided.

1.7.6 *Longitudinal Section of the Tunnels*

The longitudinal section of the tunnel has to generally conform to the upper levels of the upstream side of undersluice bay floor. If the crest of the undersluice is higher than the upstream floor, the tunnels have reverse slopes near their tail ends. In such cases, in order to obtain accelerating flow, the height of the tunnels at the exit end is reduced as compared to the inlet end. But now-a-days, the crest of the undersluice bays in which the excluder tunnels are provided is lowered to the upstream floor level and the downstream glacis slope is modified by flattening it.

1.7.6.1 The diaphragm slab is provided with an elliptical transition in the vertical plane at the inlet.

1.7.7 *Cross-section of Tunnels*

The variations of cross-section are achieved by changing width in plan. These have to be calculated for each tunnel separately and necessarily involve trial and error solutions. In some cases, changes in the height of the tunnels may be necessary as discussed in para 1.7.6.

1.7.8 *Longitudinal and Side Projection of the Diaphragm Slab over a Tunnel*

The top slab of the excluder, besides being bell mouthed, is projected beyond the noses of the tunnel divide walls by 0.3 to 1 m. The slab for each tunnel is also usually projected on the side as a cantilever. Generally these provisions are adopted on the basis of experimental evidence of improved performance.

1.7.9 *Values of Coefficient of Friction*

It is not necessary to make the floor inside the tunnels smooth, as most of the time it would be covered with coarser particles of sediment. For calculating the frictional loss through the tunnels the value of Manning's '*n*' should be chosen for this condition. The value of '*n*' usually adopted for shingle is of the order of 0.016.

1.7.10 For the structural design of silt excluders, load due to water of the pond and load due to the silt deposited inside and out side the tunnel would be considered. The worst combination of the loadings and moments, with one or more tunnels considered empty at a time and all vertical loads including the load of water inside the other tunnels should be considered.

1.7.10.1 The silt excluder has to be checked for floatation and sliding also with no silt load on top of the tunnels and no water inside all the tunnels.

1.8 Regulation Schedules

The schedule for operation of the excluder should be prepared after careful study. The tentative schedule prepared can be modified later in light of the experience gained under actual operating conditions.

1.8.1 The most commonly adopted regulation arrangement is the 'Wedge' type wherein the undersluice bay containing the excluder is the last to be opened.

1.9 Efficiency of Silt Excluder

The efficiency of a silt excluder can be expressed as

$$\eta = \frac{(qs)_1 - (qs)_2}{(qs)_1}$$

where $(qs)_1$ is the sediment proportion in the canal without excluder, and $(qs)_2$ is the sediment proportion in the canal with excluder.

1.9.1 Studies have indicated that the efficiency increases with increasing escape discharge through the excluder up to a certain limit, but than starts falling. The reason for this is that bringing in more escape discharge also means bringing in additional quantities of sediment. If even a small part of this increased quantity of sediment goes into the canal, the efficiency will be adversely affected.

1.10 Effect of Silt Excluders on the Regime of the Canal

It is always necessary to consider that grade or quantity of silt that is suited to the regime of the canal and regulate the silt entry accordingly. If the excluders are designed properly and operated judiciously. progressive silting up of the canal can be stopped and the canal restored back to its designed capacity.

1.11 Comparison between Silt Excluders and Ejectors

Sometimes, a question may arise whether to provide silt excluder or ejector or both. A comparison between the two is given in Table 1.2.

1.11.1 In light of merits and demerits of the excluder and ejector, the suitability of each for the particular work has to be examined and adopted accordingly. In heavily silt laden rivers, there may be a necessity for provision of both as in the case of Kosi Barrage.

1.12 For typical design of shingle excluders, the following design report may be referred :

"Design Report on Nangal Dam, Bhakra Nangal Project, (Chapter XI)—Design of Shingle Excluder." Published in 1954 by the Irrigation Branch, PWD, Punjab.

TABLE 1.2

Comparison between Silt Excluder and Ejector

Sl. No.	Silt Excluder	Silt Ejector
1.	The work of exclusion (preventive action) is heavy as it is subjected to river action.	The work of extraction (curative action) is light as it is subjected to only canal discharge.
2.	Exclusion is secured only once.	Multiple extractions are possible by increasing the number of ejectors.
3.	Good approach conditions are not easily obtained.	Good approach conditions in the canal are easily obtained.
4.	Canal head regulators are not required to be widened beyond their normal capacity.	Canal head regulators are required to carry extra discharge also meant for escapage.
5.	Working head for operation of the excluder is always available due to the pond.	When the head regulator supplies are decreased, working head cannot be obtained.
6.	Exclusion work is concentrated at one place.	Separate outfall works leading to the river or natural drainage near-by are necessary which even determine the location of the extractor.
7.	The approach channel being large, the excluder is not normally liable to be blocked.	The orifices of the ejector being small, they are likely to be choked by sunken debris and need a trash rack.
8.	On the whole, only slight extra cost may be involved.	On the whole, the cost of ejector may be less than that of the excluder.

CHAPTER 2

River Training Works

2.0 Introduction

In all diversion structures, some form of river training works are provided. Their objects are to (*i*) prevent outflanking of the structures, (*ii*) guide the river to flow axially uniformly through the structure, (*iii*) minimise possible cross-flows through the structure which may endanger its safety and the protection works, (*iv*) prevent flooding of the riverine lands upstream of the structure, and (*v*) provide favourable curvature of flow at the head regulator from the point of sediment entry into the canal. The problem of training the river in the boulder areas is not as serious as in plane area.

2.0.1 The different river training works generally provided are (*i*) Guide bunds, (*ii*) Afflux bunds, (*iii*) Approach Embankments, (*iv*) Groynes or spurs, (*v*) cut-offs, (*vi*) Pitching of banks and provision of launching aprons, (*vii*) Pitched islands, and (*viii*) Sills.

2.0.2 River training is a difficult operation and requires a good knowledge of the behaviour of the river at various stages of flow. If the river training works are wrongly planned, there is likelihood of damages to the costly structure even quickly. Hydraulic river model tests offer great help in the correct design of river training works. A well designed river control system constructed on a permanent basis, though costly in the first instance, proves much cheaper in the long run than resorting to temporary expedients from year to year.

2.0.3 Based on the experience gained over the years, certain parameters of the river training works can be worked out for their layout. However, these would need to be finalised with the help of three-dimensional hydraulic model tests for their adequacy in providing a satisfactory performance.

2.0.4 A typical layout of river training works is shown in Figure 2.1.

2.1 Classification of River Training Works

According to the purpose of the river training works, they can be classified into (*i*) high water training, (*ii*) low water training, and (*iii*) mean water training.

2.1.1 High water training is related to the maximum floods and aims at giving a satisfactory alignment and height of afflux bunds for disposal of floods and also includes river channel improvement. Hence they are called training for discharge.

2.1.2 Low water training aims at providing minimum depth of water for navigation during low water season. Groynes are generally used for this purpose by contracting the widths of the river channel. This class of river training can be called training for depth.

2.1.3 Alteration in the river cross-section and alignment must be done in accordance with that stage of the river at which the maximum movement of sediment takes place over a period of years. Only in the mean stage between the high and low stages, the combined effect of forces causing sediment movement and the time for which such forces are maintained is maximum. This stage influences the configuration of the river. Training works planned for this stage can be called training for sediment.

2.2 Guide Bunds

Alluvial rivers in flood plains spread over a very large area during floods and it would be very costly to provide bridges or any other structure across the entire natural spread. It is necessary to narrow down and restrict its course to flow axially through the diversion structure. Guide bunds are provided for this purpose of guiding the river flow past the diversion structure without causing damage to it and its approaches. They are constructed on either or both on the upstream and down stream of the structure and on one or both the flanks as required.

2.2.1 *Classification of Guide Bunds*

Guide bunds can be classified according to their form in plan as (*i*) divergent, (*ii*) convergent, and

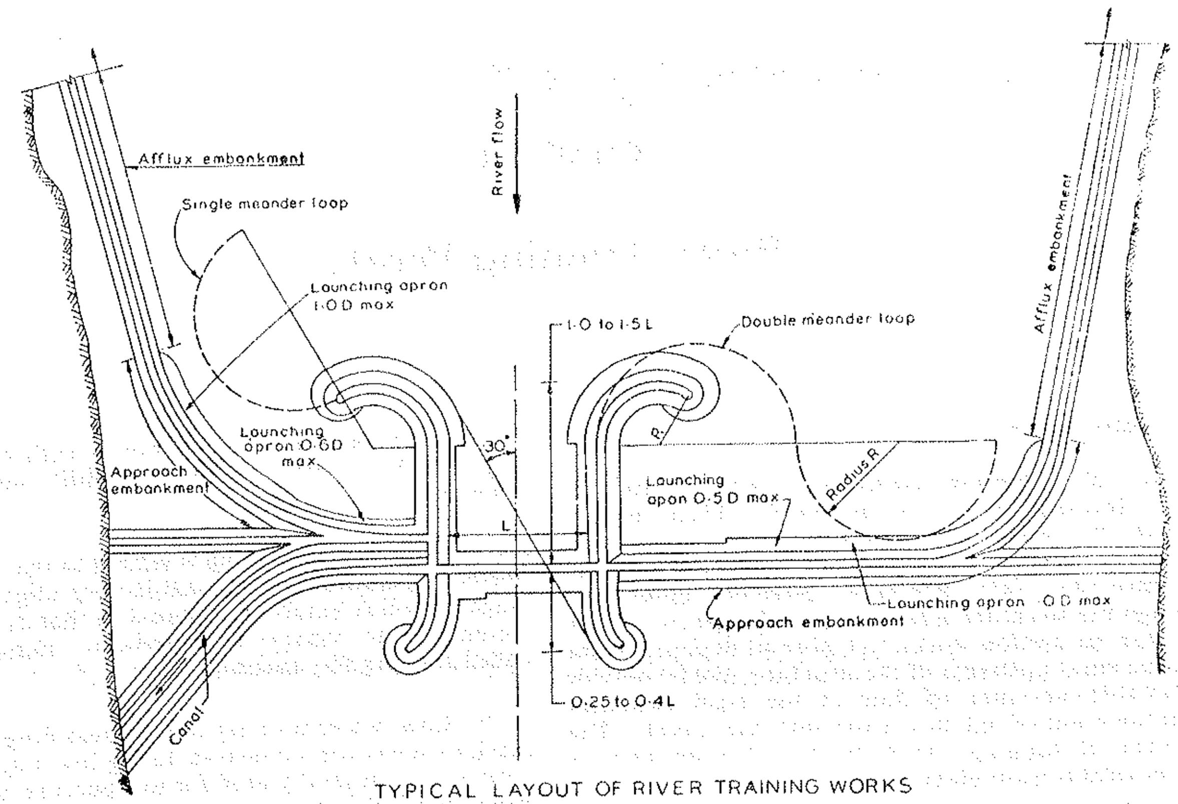


FIGURE 2.1

(*iii*) parallel and according to their geometrical shape as straight and elliptical with circular or multi-radii curved head. These are shown in Figure 2.2.

2.2.1.1 In the case of divergent guide bunds, the approach embankment gets relatively less protection under worst possible embayment and hence divergent guide bunds require a longer length for the same degree of protection as would be provided by parallel guide bunds. They also induce oblique flow on to the diversion structure and give rise to tendency of shoal formation in the centre due to larger waterway between curved heads. However, in the case of oblique approaching flow, it becomes obligatory to provide divergent guide bunds to keep the flow active in the spans adjacent to them.

2.2.1.2 The convergent guide bunds have the disadvantage of excessive attack and heavy scour at the head and shoaling all along the shank rendering the end bays inactive.

2.2.1.3. Parallel guide bunds with suitable curved head have been found to give uniform flow from the head of guide bunds to the axis of the diversion structure.

2.2.1.4 In the case of elliptical guide bunds, due to gradual change in the curvature, the flow is found to hug the bunds all along their lengths whereas in the case of straight guide bunds, separation of flow is found to occur after the curved head, leading to obliquity of flow. Elliptical guide bunds have also been found to provide better control on development and extension of meander loop towards the approach embankment.

2.2.2 *Length of Guide Bunds*

The length of the guide bund on the upstream is generally kept as 1·0 to 1.5 L where L is the width between the abutments of the diversion structure. In order to avoid heavy river action on the guide bunds, it is desirable to limit the obliquity of flow to the river axis not more than 30° as indicated in Figure 2.1. The length of the downstream guide bund is kept as 0.25 L to 0.4 L (Figure 2.2).

2.2.2.1. For wide alluvial belt, the length of guide bunds is decided from two important considerations, viz., the maximum obliquity of the current and the permissible limit to which the main channel of the river can be allowed to flow near the approach embankment in the event of the river developing excessive embayment behind the training works. The radius of the worst possible loop has to be ascertained from the data of the acute loops formed by the river during the past. Where river survey is not available, the radius of the worst loop can be determined by dividing the radius of the average loop worked out from the available surveys of the river by 2.5 for rivers having a maximum discharge up to 5,000 cumecs and by 2.0 for a maximum discharge above 5,000 cumec.

2.2.3 *Curved Head and Tail of Guide Bunds*

2.2.3.1 The upstream curved head guides the flow

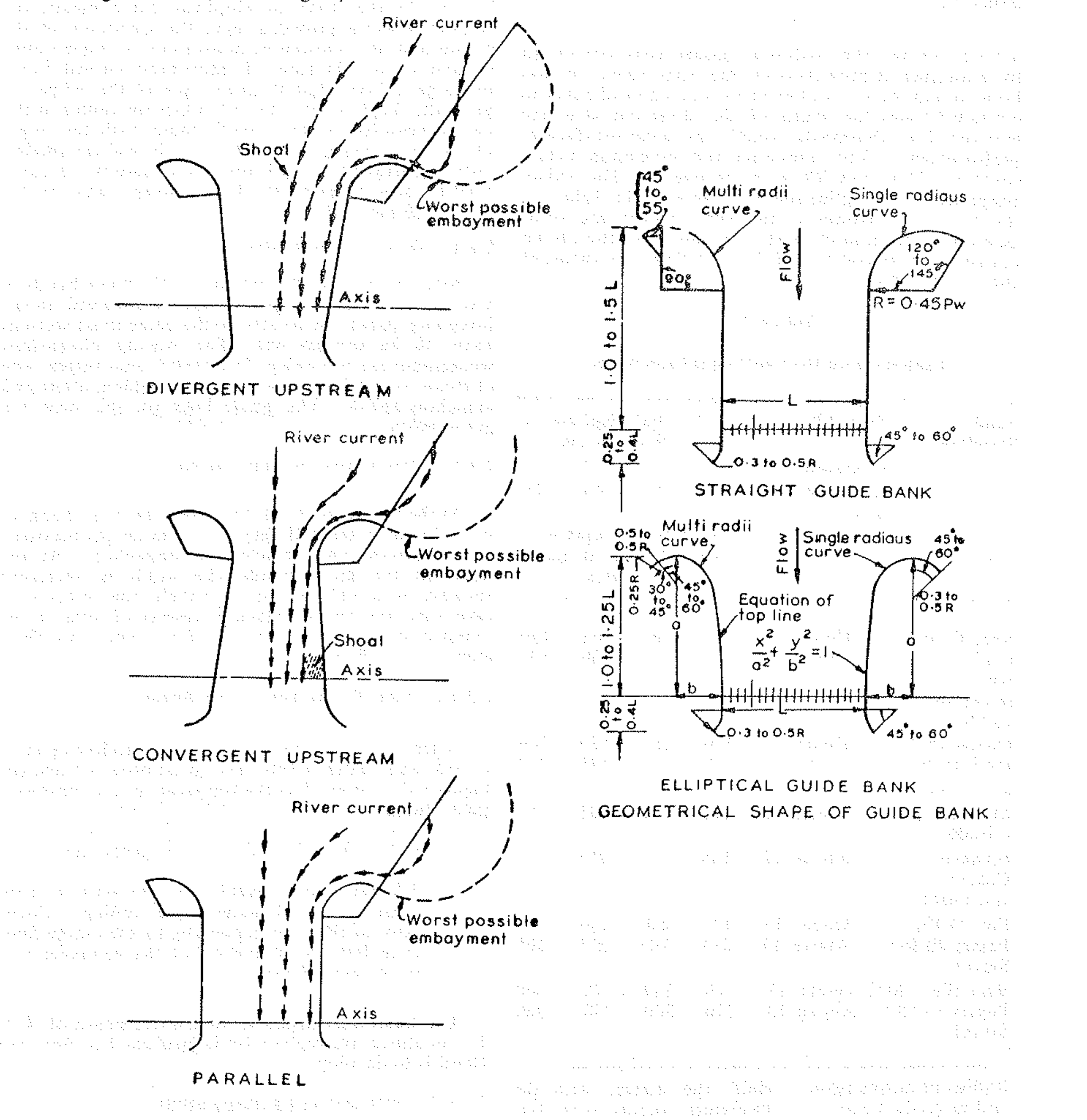


FIGURE 2.2 : Types and General Dimensions of Guide Bund.

smoothly and axially to the diversion structure keeping the end spans active. The radius of the curved head should be kept as small as possible consistent with the proper functioning of the guide bund. The downstream curved tail provides a smooth exit of flow from the structure.

2.2.3.2 From the hydraulic model tests conducted for a number of projects over the past years, it has been found that a radius of the curved head equal to 0.4 to 0.5 times the width of the diversion structure between the abutments usually provides satisfactory performance. The minimum and maximum values could be 150 m and 600 m respectively. The values suggested by Spring for the radii are given in Table 2.1 for guidance. However, the exact values are to be ascertained from model tests. The radius of the curved tail generally ranges from 0.3 to 0.5 times the radius of the curved head.

TABLE 2.1

Radii of Curved Head and Tail of Guide Bunds

Sand Standard	Probable Maximum Abnormal scour in metres	Fall per km of River in cm: 9	14	19	28
		Radius of upstream curved end of guide bank in metres			
Very Coarse (25% of the whole retained by 160, IS : Sieve.	Under 6	75	90	105	120
	Above 6	90	110	130	150
Coarse (80% retained by 60 IS : Sieve)	Under 9	110	125	150	200
	Above 9	130	155	175	210
Medium (Grade between Coarse and Fine).	Under 12	125	165	185	255
	Above 12	165	195	225	255
Fine (80% Passes 20 IS : Sieve)	Under 15	175	200	230	255
	Above 15	215	245	275	305
Very Fine (20% Passes 15 IS : Sieve)	Under 18	210	240	270	300
	Above 18	270	300	330	360
Radius of downstream end of guide bund	Half the above, with the minimum radius such that maintenance trains can be shunted.				

2.2.3.3 According to the river curvature, the angle of sweep of curved upstream head ranges from 120° to 145°. The angle for the curved tail usually varies from 45° to 60°.

2.2.3.4 In the case of elliptical guide bunds, the elliptical curve is provided upto the quadrant of the ellipse and is followed by multi-radii or single radius circular curve. In case of multi-radii curved head, the larger radius adjacent to the apex of the ellipse is generally kept as 0.3 to 0.5 times the radius of the curved head for straight guide bund with the angle of sweep varying from 45° to 60° and the smaller radius equivalent to 0.25 times the radius of curve head for straight guide bund with sweep angle of 30° to 40° (Figure 2.2.)

2.2.4. *Design of Guide Bunds*

After fixing up the layout of the guide bunds in accordance with the guide lines mentioned in the foregoing paras, the details of the guide bund sections have to be worked out. The various dimensions worked out are top width, free board, side slopes, size of stone for pitching, thickness of pitching, filters and launching apron. The guide lines for the same are given below.

2.2.4.1 TOP WIDTH OF GUIDE BUND

At the formation level, the width of the shank of guide bunds is generally kept 6 to 9 m to permit carriage of material and vehicles for inspection. At the nose of the guide bunds, the width is increased suitably in a bulb shape to enable the vehcles to take turn and also for stacking reserve of stone to be dumped in places whenever the bunds are threatened by the flow.

2.2.4.2 FREE BOARD FOR GUIDE BUND

A free board of 1 to 1.5 m above the following mentioned two water levels has to be provided and the higher value adopted as the top level of the upstream guide bund :

(*i*) Highest flood level for 1 in 500years flood.

(*ii*) Affluxed water level in the rear portion of the guide bank calculated after adding velocity head to HFL corresponding to the design flood (1 in 100 year frequency) at the upstream nose of the guide bank.

On the downstream side also, a free board of **1** to 1.5 m above the highest flood level for 1 in 500 years flood is to be adopted.

2.2.4.3. SIDE SLOPES OF GUIDE BUND

The side slopes of the guide bund have to be fixed from stability considerations of the bund which depend on the material of which the bund is made and also its height. Generally the side slopes of the guide bund vary from 2 : 1 to 3 : 1 (H : V).

2.2.4.4 SIZE OF STONE FOR PITCHING

The sloping surface of the guide bund on the water side has to withstand erosive action of flow. This is achieved by pitching the slope manually with stones. The size and weight of the stones can be approximately determined from the curves given in Figure 2.9. It is desirable to place the stones over filters so that fines do not escape through the interstices of the pitching. For average velocities up to 2 m/sec, burnt clay brick on edge can be used as pitching material. For an average velocity upto 3.5 m/sec, pitching of stone weighing from 40 to 70 kg (0.3 to 0.4 m in diameter) and for higher velocities, cement concrete blocks of depth equal to the thickness of pitching can be used. On the rear side, turfing of the slope is normally found to be adequate.

2.2.4.5 THICKNESS OF PITCHING

The thickness of pitching is to be kept equal to the size of the stone for pitching determined as mentioned in para 2.2.4.4. However, it should not be less than 0.25 m. Wherever the velocities are high for which the size of stone is greater than 0.4 m, cement concrete blocks of thickness 0.4 to 0.5 or 0.6 m may be used.

2.2.4.6 PROVISION OF FILTER

It is always desirable to provide an inverted (graded) filter below the pitching stones to avoid the finer bund materials getting out through the interstices. The thickness of the filter may be 20 to 30 cm. Filter has to satisfy the criteria with respect to the next lower size and with respect to the base material :

(*i*) For uniform grain size filter,

$$R_{50} = \frac{D_{50} \text{ of filter material}}{D_{50} \text{ of base material}} = 5 \text{ to } 10$$

(*ii*) For graded material of sub-rounded particles,

$$R_{50} = \frac{D_{50} \text{ of filter material}}{D_{50} \text{ of base material}} = 12 \text{ to } 58$$

$$R_{15} = \frac{D_{15} \text{ of filter material}}{D_{15} \text{ of base material.}} = 12 \text{ to } 40$$

2.2.4.7 LAUNCHING APRON

Just as launching apron is provided for the main structure both on the upstream and downstream it has to be provided for guide bunds also in the bed in continuation of the pitching. The different aspects to be looked into are the size of the stones, depth of scour, thickness, slope of launched apron, shape and size of launching apron.

The required size of stone for the apron can be obtained from the curves given in Figure 2.9. In case of non-availability of required size of stones, cement concrete blocks or stone sausages, prepared with 4 mm GI wire in double knots and closely knit and securely tied, may be used.

The scour depths to be adopted in the calculations for the launching apron would be different along the length of the guide bund from upstream to downstream. These are given below in Table 2.2. Reference may be made to Chapter 6 of Vol. I on Hydraulic Designs for Estimation of Scour Depth.

TABLE 2.2

Scour Depths for Design of Guide Bund Apron

Location	Maximum Scour Depth to be Adopted
Upstream Curved Head of Guide Bund	2.5 *D*
Straight Reach of Guide Bund to Nose of Downstream Guide Bund	1.5 *D*

While calculating the scour values, the discharge corresponding to 50 to 100 years frequency may be adopted. However, after construction and operation of the diversion structure, the portions of the guide bund coming under attack of the river flow should be carefully inspected and strengthened as and when necessary.

The thickness of apron of the guide bund should be about 25 to 50 percent more than that required for the pitching. While the slope of the launched apron for calculation of the quantity can be taken as 2 : 1 for loose boulders or stones, it may be taken as 1.5 : 1 for c.c. blocks or stone sausages.

From the behaviour of the guide bunds of previously constructed diversion structures, it has been observed that shallow and wide aprons launch evenly if the scour takes place rapidly. If the scour is gradual, the effect of the width on the launching of apron is marginal. Generally a width of 1.5 *D* has been found to be satisfactory. For the shank or straight portions of the guide bunds, the thickness of the apron may be kept uniform at 1.5 T where T is the thickness of the stone pitching. To cover a wider area, for the curved head, the thickness is increased from 1.5 to 2.25 T with suitable transition over a length of L_1 equal to one fourth of the radius of the curved head and provided in the shank portion only. On the rear side of the curved head and nose of the guide bund, the apron should be turned and ended in a length equal to about one fourth of the respective radius.

The details of the various dimensions for the guide bunds are depicted in Figure 2.3.

2.2.5 Typical layout and sections of upstream and downstream guide bunds are shown in Figures 2.4 and 2.5.

2.2.6 Salient dimensions of guide bunds of different projects are given in Table 2.3.

2.3 Afflux Bund

Afflux bunds extend from the abutments of guide bunds (usually) or approach bunds as the case may be. The upstream afflux bunds are connected to grounds with levels higher than the afflux highest flood level or existing flood embankments, if any. The downstream afflux bunds, if provided, are taken to such a length as would be necessary to protect the canal/approach bunds from the high floods.

Afflux bunds are provided on upstream and downstream to afford flood protection to low lying areas as a result of floods due to afflux created by the construction of bridge/structure and to check outflanking the structure.

2.3.1 *Layout of Afflux Bund*

The alignment of the afflux bund on the upstream usually follows the alluvial belt edge of the river if the edges are not far off. In case the edges are far off, it can be aligned in alluvial belt, but it has to be ensured that the marginal embankment is aligned away from the zone of high velocity flow. Since the rivers change their course, it is not necessary that a particular alignment safe for a particular flow condition, may be safe for a changed river condition. Hence the alignment satisfactory and safe for a particular flow condition (constructed initially) has to be constantly reviewed after every flood and modified, if necessary.

2.3.2 *Top Width of Afflux Bund*

Generally the top width of the afflux bund is kept as 6 to 9 m at formation level.

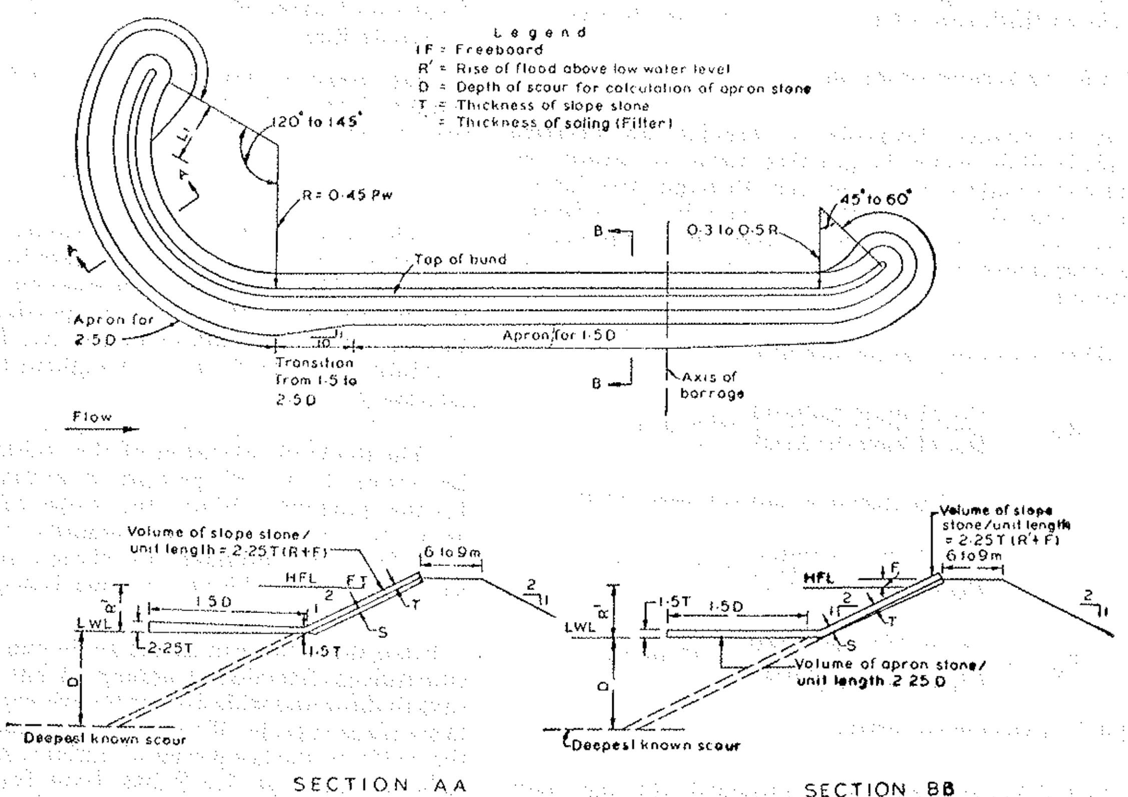


FIGURE 2.3 : Details of Guide Bund

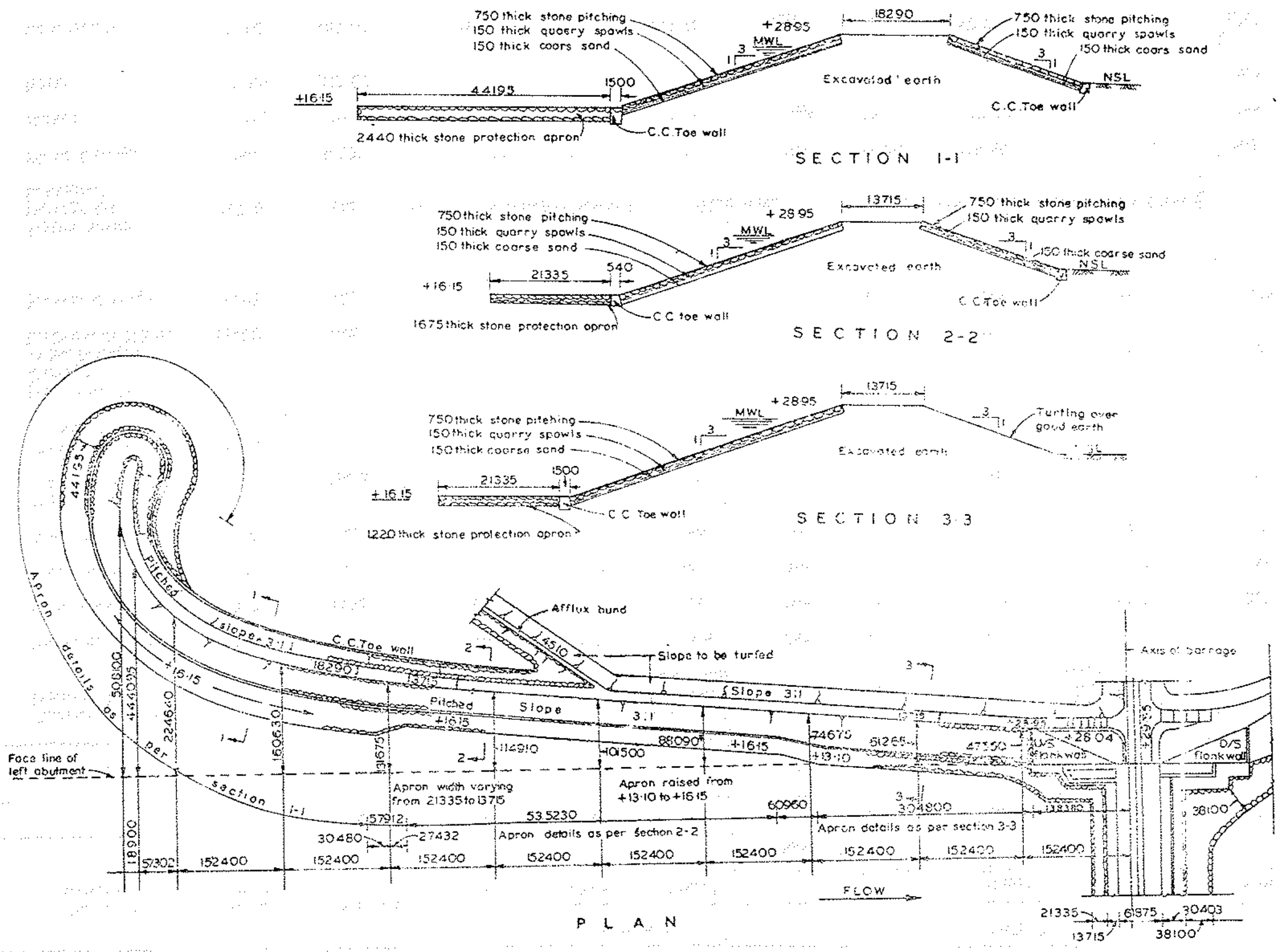


FIGURE 2.4 : Typical Layout and Sections of Upstream Guide Bund.

TABLE 2.3

Salient Dimensions of Guide Bunds of Projeets

				Up stream Guide Bund							
				Right Bank				Left Bank			
Sl. No.	Name of structure	Design discharge cumecs	Water way provided	Projected Length	Straight shank	Radius of curved head	Sweep Angle	Projected Length	Straight Shank	Radius of Curved head	Sweep Angle
			M	M	M	M		M	M	M	M
1	2	3	4	5	6	7	8	9	10	11	12
1.	Gandak Barrage Project (Bihar)	24100	742	998	—	Guide bund continued as afflux bund	—	570	—	Guide bund continued as Afflux Bund	—
2.	Yamuna (Delhi)	8500	553	331	226	75.6	60°	—	—	—	—
3.	Kosi (Bihar)	27000	1150	1025	457	518	90°	644	457	152.5	120°
4.	Wazirabad (Delhi)	7081	—	613	56.7	84	90°	81	—	51.8	90°
5.	Sone (Bihar)	41000	1407	885	427	480	120°	625	183	480	75°
6.	Girija Barrage (i) For Hydraulic design (ii) For Free Board	22 200 26500	716.0	955	Nil	305	30°	547	425	122	120°
7.	Lower Sarda Barrage at Sardanagar Lakhimpur Kheri	11400	408	693.99	Nil	232	30°	428	Nil	232	30°
8.	Narora Barrage	16980	906					792	492	300	90°
9.	Upper Sarda Barrage (at Banbassa)	16980	605	Straight bells bund combined with afflux bund				(Straight bell bund with no curved head)			
10.	Phika Barrage	1048	107.90	129.54	53.34	76.20	120°	215.80	139.60	76.20	120°
11.	Kemri	1982	162.76	—	87.48	91.44	60°	178.92	87.48	91.44	120°
12.	KHO	2830	203.00					30,50	—	30.50	90°
13.	Ram Ganga	7360	408.00	582.71	362.71	219.46	120°	582.17	362.71	219.46	120°
14.	Dakpathar	14447	516.33	685.80	563.88	121.92	90°	—	624.84 (TOE)	220.98	60°
15	Ahsan	4500	287.73	290.00	165.00	125.00	90°	358.00	Joined with high bank 210.00	200.00 100.00	50°&40

Down Stream GuideBund									
Right Bank				Left Bank					
Projected Length	Straight Shank	Radius of Curved Head	Sweep Angle	Projected Length	Straight Shank	Radius of Curved Head	Sweep Angle	REMARKS	
M	M	M		M	M	M			
13	14	15	16	17	18	19	20	21	
185.8	48.8	91.5	90°	185.8	48.8	91.5	90°		
200	61.6	100	100°	—	—	—	—		
192	—	152.5	90°	192	—	152.5	90°		
48	—	20	75°	54.3	—	30.5	90°		
183	91,5	122	65°	183	91.5	122	65°		
161.4	75	122	45°	133.3	47.1	122	45°	U/S/(Right) Divergent U.S. Elliptical with single Radius curved head	U/S/ (Left) Parallel straight with single Radius curved head
93.75	40.75	75	45°	91.25	38.25	75	45°	Elliptical with single radius curved head	Elliptical with single radius curved head
300	173	180	45°	300	173	180	45°	Canal head Regulator and Colony (no Guide Bunds on RT Bank)	Bell Tupe
No Guide Bund (Masonry Core walls provided)									
53.11	25.68	27.43	120°	53.11	25.68	27.43	120°	PARALLEL	PARALLEL
54.86	36.57	18.29	90°	54.86	36.57	18.29	90°	,, ,,	,, ,,
75.99	54.42	30.50	45°	—	23.92	39.00	58°	Joined with circular curved head with right abutment of bridge at 0.2 km upstream.	
86.24	32.36	76.20	45°	86.26	32.36	76.20	45°	PARALLEL	PARALLEL
202.69	111.25	91.44	90°	202.69	111.25	91.44	90′	,, ,,	,, ,,
175.00	100.00	75.00	90°	153.00	100.00	75.00	45°	,, ,,	Parallel with Multi Radii

TABLE 2.3 (*Contd*)

1	2	3	4	5	6	7	8	9	10	11	12
16.	Kosi	5100	138.50	—	287.00	52.00	43°	108.20(TOE)	—	40.00	30°
17.	Harike (Punjab)	15010	636.12	—	438	253.59	60°	—	586.74	190.5	60°
18.	Ferozpur (Punjab)	12745	596.19	—	822.96	—	0°	—	822.96	—	0°
19.	Ropar Madhopur Nangal	No Guide Banks or marginal bunds are provided, as these Barrages have natural high ground as its abutments									Straight
20.	Hasdeo Barrage (M P.)	19822	14 Bays of 18.287-M	180 metre from Barrage axis at 30° to divide wall	120 metre from Barrage axis at 30° to divide wall	9 metre Semi-circular	30° to divide wall	1030 M	80 M	It is not Uniform	850 Merte Throughout curved
21.	Khodashi weir across Krishna River -New Bombay	66830 Cusecs i.e. 1891.29/ cum/Sec.	1133 ft 345.34 meters (i.e. length of weir)	3200 ft. measured length as record is not available	140 ft on U/S	735 ft		not provided			
22.	Pick-up weir on Mahi river at Wanakbori (Gandhinagar)	31130 M^3/Sec.	674 M	38.43 Mts	40.26 Mts	50.63 Mts.	134°	30.95 Mts.	—	29 M	180°
23.	Wasna Barrage	21238 M^3/ Sec.	548.64	457.20 M	182.88 M	243.84 M	69°	1554..50 M	1554.50 M	—	—

TABLE 2.3 (*Contd.*)

13	14	15	16	17	18	19	20	21	
70.48	57.75	18.00	45°	57.73	45.00	18.00	45°	Diverging joined with High Bank on right	Elliptical
—	243.84	—	0°	—	182.88	—	0°	PARALLEL	Diverging
—	201.17	—	0°	—	201.17	—	0°	,,	PARALLEL
			Straight				Straight		
Nil	10 M	Not Uniform						Rockfill with soft core	Earth fill
90' Masonry no D/S guide bund (only flood protection on wall of Khodashi village of 90' D/S from weir (Straight) is provided)					No Guide Bund (only masonry wall below H.F.L. is provided to protect the bank near Trolly shed)			Earthan with U/S Dry Rubble Pitching	
76.25 Mts.	—	straight	180°	60.24 Mts.	—	Straight	180°	76 cms Thick Rubble Pitching	Masonry Wall
—	—	—	—	—	—	—	—	Earthen Dam with C.O.T. and impervious zone	Earthen Dam with C.O.T. and impervious zone

2.3.3. *Free Board for Afflux Bund*

The top level of the afflux bund is fixed by providing a free board of 1 to 1.5 m over the affluxed highest flood level for a flood of 1 in 500 years frequency.

2.3.4. *Slope Pitching and Launching Apron*

Generally, the afflux bunds are constructed away from the main channel of the river. Hence they are not usually subjected to strong river currents. In such cases, provision of slope pitching and launching apron are not considered necessary. However, it is desirable to provide a vegetal cover or turfing. In reaches where strong river currents are likely to attack the afflux bunds, the slopes may be pitched as for the guide bunds.

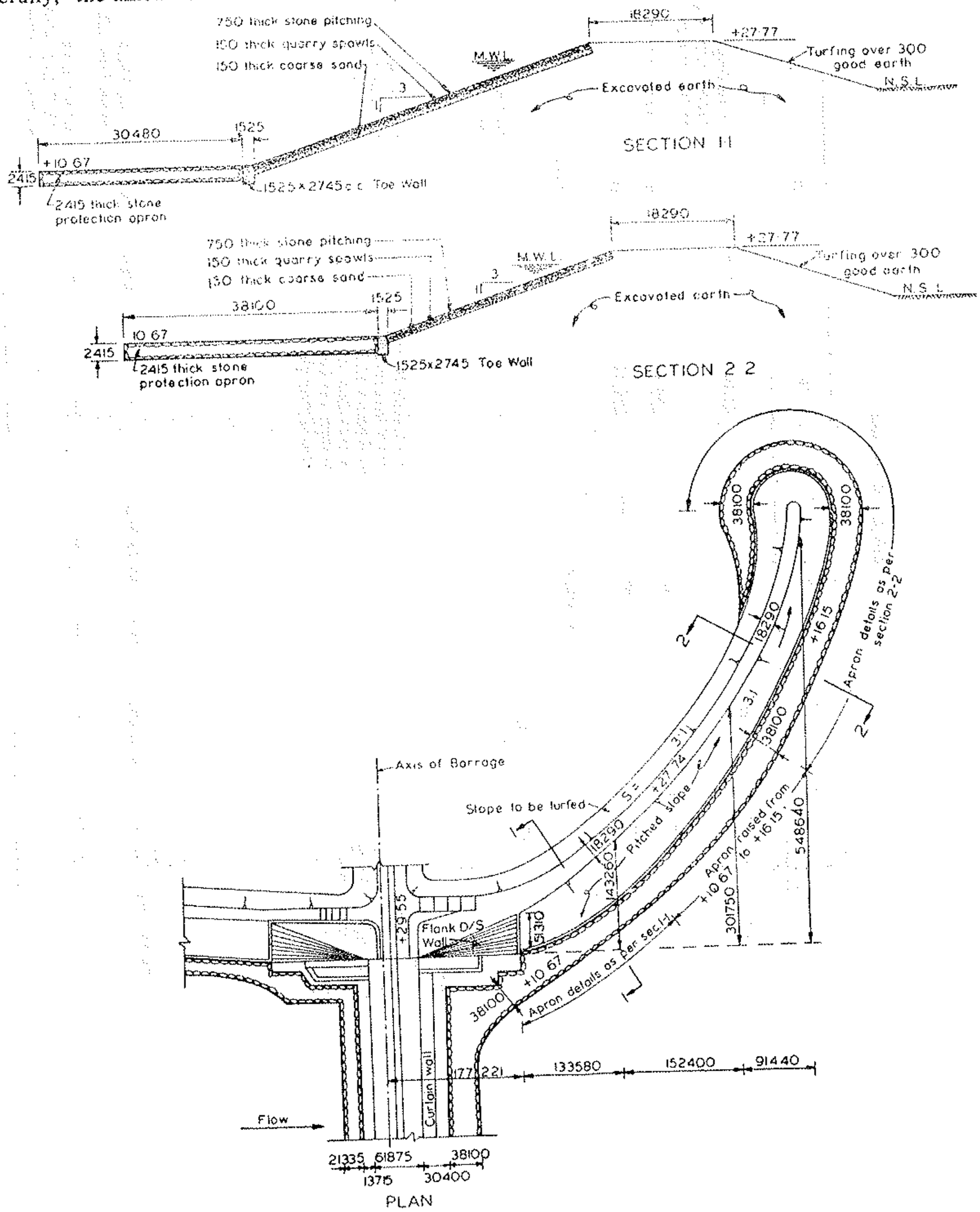


FIGURE 2.5 : Typical Layout and Sections of Downstream Guide Bund.

2.3.5 Typical layout and section of afflux bund are shown in Figure 2.6.

2.4. Approach Embankments

Where the width of the river is very wide in an alluvial plain, the diversion structure is constructed with a restricted waterway for economy as well as better flow conditions. The unbridged width of the river is blocked by means of embankments called Approach embankments or tie bunds.

2.4.1. *Layout of Approach Embankment*

In case of alluvial plains, the river forms either a single loop or a double loop depending upon the distance between the guide bunds and the alluvial belt edges. Hence the approach embankments on both the flanks should be aligned in line with the axis of the diversion structure up to a point beyond the range of worst anticipated loop. Sometimes the approach embankments may be only on one flank depending on the river configuration.

2.4.2. *Top Width of Approach Embankment*

The top width of the approach embankment is usually kept as 6 to 9 m at formation level.

2.4.3. *Free Board of Approach Embankment*

Free board for the approach embankment will follow the guide lines given for guide bunds in para 2.2.4.2.

2.4.4. *Side Slopes of Approach Embankment*

The side slopes of the approach embankment will follow the guide lines given for guide bunds in Para 2.2.4.3. On the canal side, the section of the embankment would be designed for the condition of high flood level on the upstream side and no water on the downstream side. For the other side, *i.e.*, where canal is not laidout, the design condition would be affluxed high flood on the upstream and retrogressed water level on the downstream.

2.4.5. *Size of Stone for Pitching*

Velocities for 40 percent of the design discharge would be estimated and the size of stone for pitching would be determined as for guide bunds given in para 2.2.4.4.

2.4.6. *Thickness of Pitching*

The Guide lines for determining the thickness of pitching would be the same as for guide bunds mentioned in para 2.2.4.5. The velocities would be estimated for 40 percent of the design discharge.

2.4.7. *Provision of Filter*

Generally filters are not provided below the pitch-

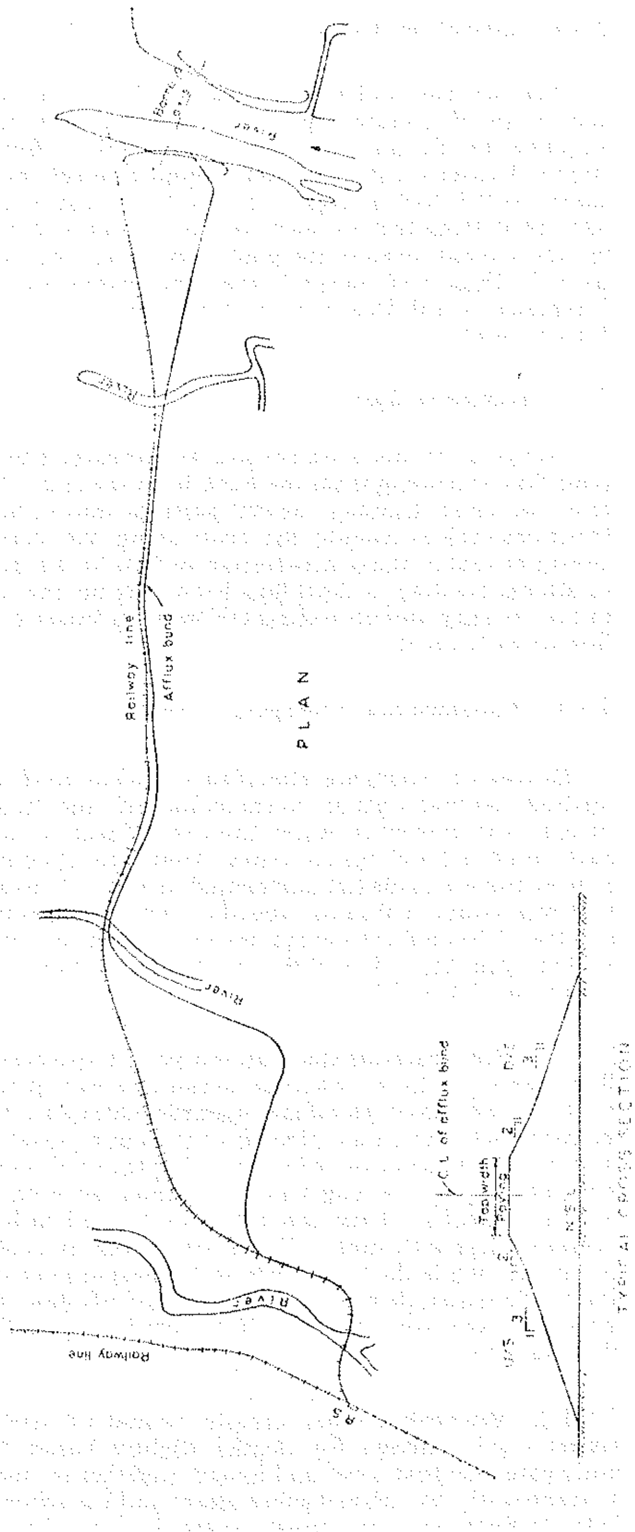


FIGURE 2.6 : Typical Alignment and Section of Afflux Bund.

ing stones in the case of approach embankments. However, if the section of embankment is heavy, filter may be provided as mentioned for guide bunds in para 2.2.4.6.

2.4.8. *Launching Apron*

The provisions of size of stone, thickness of apron and slope of launched apron would be similar to those of guide bunds mentioned in para 2.2.4.7. But the depth of scour for the approach embankment may be taken as 0 5. to 1.0. D_{max}. The width of the launching apron from the abutment to the distance covered by the curved head of the guide bund may be taken as 0 5. D_{max} and beyond that the width may be increased to 1 0 D_{max} with suitable transition in the former reach.

2.5. **Groynes or Spurs**

Groynes or spurs are constructed transverse to the river flow extending from the bank in to the river. This form of river training works perform one or more functions such as training the river along the desired course to reduce the concentration of flow at the point of attack, creating a slack flow for silting up the area in the vicinity and protecting the bank by keeping the flow away from it.

2.5.1. *Classification of Groynes or Spurs*

Groyes or spurs are classified according to (*i*) the method and materials of construction (*ii*) the height of spur with respect to water level (*iii*) function to be performed and (*iv*) special types. These are (*i*) permeable or impermeable (*ii*) submerged or non-submerged (*iii*) attracting, deflecting repelling and sedimenting and (*iv*) T-headed (Denehey), hocky type, burma type, kinked type, etc. The different types of spurs are shown in Figure 2.7.

2.5.1.1. Impermeable spurs do not permit appreciable flow through them whereas permeable ones permit restricted flow through them. Impermeable spurs may be constructed of a core of sand or sand and gravel or soil as available in the river bed and protected on the sides and top by a strong armour of stone pitching or concrete blocks. They are also constructed of balli crates packed with stone inside a wire screen or rubble masonry. While the section has to be designed according to the materials used and the velocity of flow the head of the spur has to have special protection (Figure 2.7.).

2 5.1.2. Permeable spurs usually consist of timber stakes or piles driven for depths slightly below the anticipated deepest scour and joined together to form a framework by other timber pieces and the space in between filled up with brush wood or branches of trees. The toe of the spur would be protected by a mattress of stones or other material. As the permeable spurs slow down the current, silt deposition is induceed. These spurs, being temporary in nature, are susceptible to damage by floating debris. In bouldery or gravelly beds, the spurs would have to be put up by weighing down timber beams at the base by stones or concrete blocks and the other parts of the frame would then be tied to the beames at the base.

2.5.1.3. The head of the spur causes the current to be deflected in a diversion nearly perpendicular to itself. This current of water coming in to contact with the pool of still water area adjacent to the area causes formation of vertical boring eddies and deep scours. When the spur is inclined downstream, the scour hole is formed closer to the bank than the one caused by the spur inclined slightly upstream. Hence the former is called attracting spur and the latter repelling spur. Hence for bank protection and deflection of current away from it, either perpendicular or repelling spurs are used. The former is used mostly on convex banks.

2.5.1.4. In Denehey's groynes, cross groynes at the head are provided giving them T-shape. The cross groyne protects the main groyne on the same principle as the main groyne protects the bank.

2.5.1.5. In the Hockey spurs, their lower ends are shaped like hockey sticks. They accentuate the attracting tendencies of the spur and are not likely to be useful for bank protection or repelling of the current away from the bank (Figure 2.7.).

2.5.2. *Layout of Groynes or Spurs*

Groynes are much more effective when constructe d in series as they create a pool of nearly still water between them which resists the current and gradually accumulates silt forming a permanent bank line in course of time. The repelling spurs are constructed with an inclination upstream which varies from 10° to 30° to the line normal to the bank. In the T-shaped groynes, a greater length of the cross groyne projects upstream and a smaller portion downstream of the main groyne.

2.5.3. *Length of Groynes*

The length of groynes depends upon the position of the original bank line and the designed normal line of the trained river channel. In easily erodible rivers, too long groynes are liable to damage and failure. Hence, it would be better to construct shorter ones in the beginning and extend them gradually as silting between them proceeds. Shorter and temporary spurs constructed between long ones are helpful in inducing silt deposition.

2.5.4. *Spacing of Groynes*

Each groyne can protect only a certain length and so the primary factor governing the spacing between adjacent groynes is their lengths. Generally, a spacing of 2 to 2.5 times the length of groynes at convex banks and equal to the length at concave banks is adopted. Attempts to economise in cost by adopting wider spacings with a view to insert intermediate groynes at a later date may not give the desired results as the training of river would not be satisfactory and maintenance may pose problems and extra expenditure. T-shaped groynes are generally placed 800 m apart with the T-heads on a regular curved or straight line.

2.5.4.1. Apart from the length, the width of the river, location of the groynes and type of construction also govern the spacings. For wide rivers, the ratio of spacing to length of groyne would be larger than that for a narrow river for the same flood value. For convex banks, the spacing would be larger than that for concave banks. Intermediate values would be adopted for straight reaches. Spacing of permeable spurs would be larger than that of impermeable or solid spurs.

2.5.5. *Design of Groynes or Spurs*

The design of the groynes or spurs include the fixation of top width, free board, side slopes, size of stone for pitching, thickness of pitching, filter and launching apron.

2.5.5.1. Top Width of Spur

The top width of the spur is kept as 3 to 6 m at formation level.

2.5.5.2. Free Board

The top level of the spur is to be worked out by giving a free board of 1 to 1.5 m above the highest flood level for 1 in 500 year flood or the anticipated highest flood level upstream of the spur, whichever is more.

2.5.5.3. Side Slopes

The slopes of the upstream shank and nose is generally kept not steeper than 2 : 1 the downstream slope varies from 1.5 : 1 to 2 : 1.

2.5.5.4. Size of Stone Fore Pitching

The guide lines for determining the size of stone for pitching for guide bunds hold good for spurs also.

2.5.5.5. Thickness of Pitching

The thickness of pitching for spurs may be determined from the formula $T = 0.06\ Q^{1/3}$ where Q is the design discharge in cumecs. The thickness of stone need not be provided the same through-out the entire length of the spur. It can be progressively reduced from the nose as given in the Table 5.4.

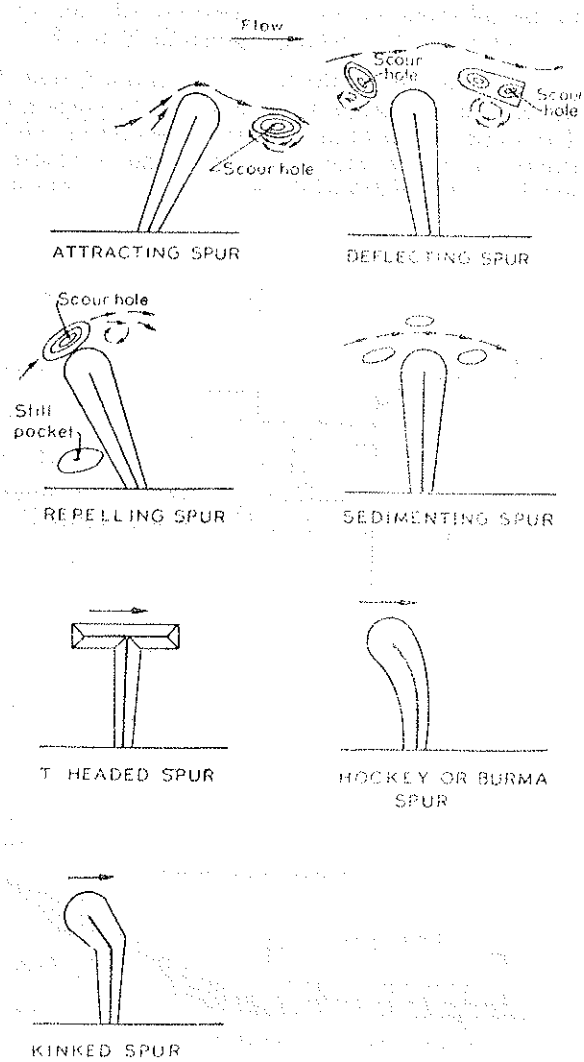


FIGURE 2.7 : Types of Spurs/Groynes.

2.5.5.6. Provision of Filters

Provision of filter satisfying the filter criteria has to be made below the pitching at nose and on the upstream face for a length of 30 to 45 m from the nose. The thickness of the same may be 20 to 30 cm. The thickness of filter for the next 30 to 45m on the upstream face may be reduced to about 15 cm and beyond that, it can be omitted.

2.5.5.7. Launching Apron for Groynes or Spurs

Just like guide bunds, launching apron has to be provided for groynes also. The important dimensions to be determined are the size of stone, depth of scour, thickness, slope of launched apron, shape and size of apron.

The size of stone for launching apron would be similar to those for guide bunds. The scour depths to be adopted in various reaches of the spur for calculation of the apron are given in Table 2.5.

The thickness of the launching apron would be 25 to 50 percent more than that for the slope pitching. The slopes of the launching apron would be similar to those for guide bunds. In the semi circular nose portion, the width of the apron would be equal to 1.5 D_{max} and would be continued up to 60 to 90 m on the upstream or for a length up to which the river action prevails (whichever is more). The width may be reduced to 1.0 D_{max} for the next 30 to 60 m on the upstream with suitable transition. For the remaining length of the spur on the upstream, nominal apron or no apron may be provided depending on the flow conditions. On the down-stream side, beyond the nose, the width would be reduced from 1.5 D_{max} to 1.0 D_{max} over a length of 15 to 30 m and continued at 1.0 D_{max} for the next 15 to 30 m. If the return flow prevails beyond the above specified reach, the apron length may be increased to cover the region of return flow.

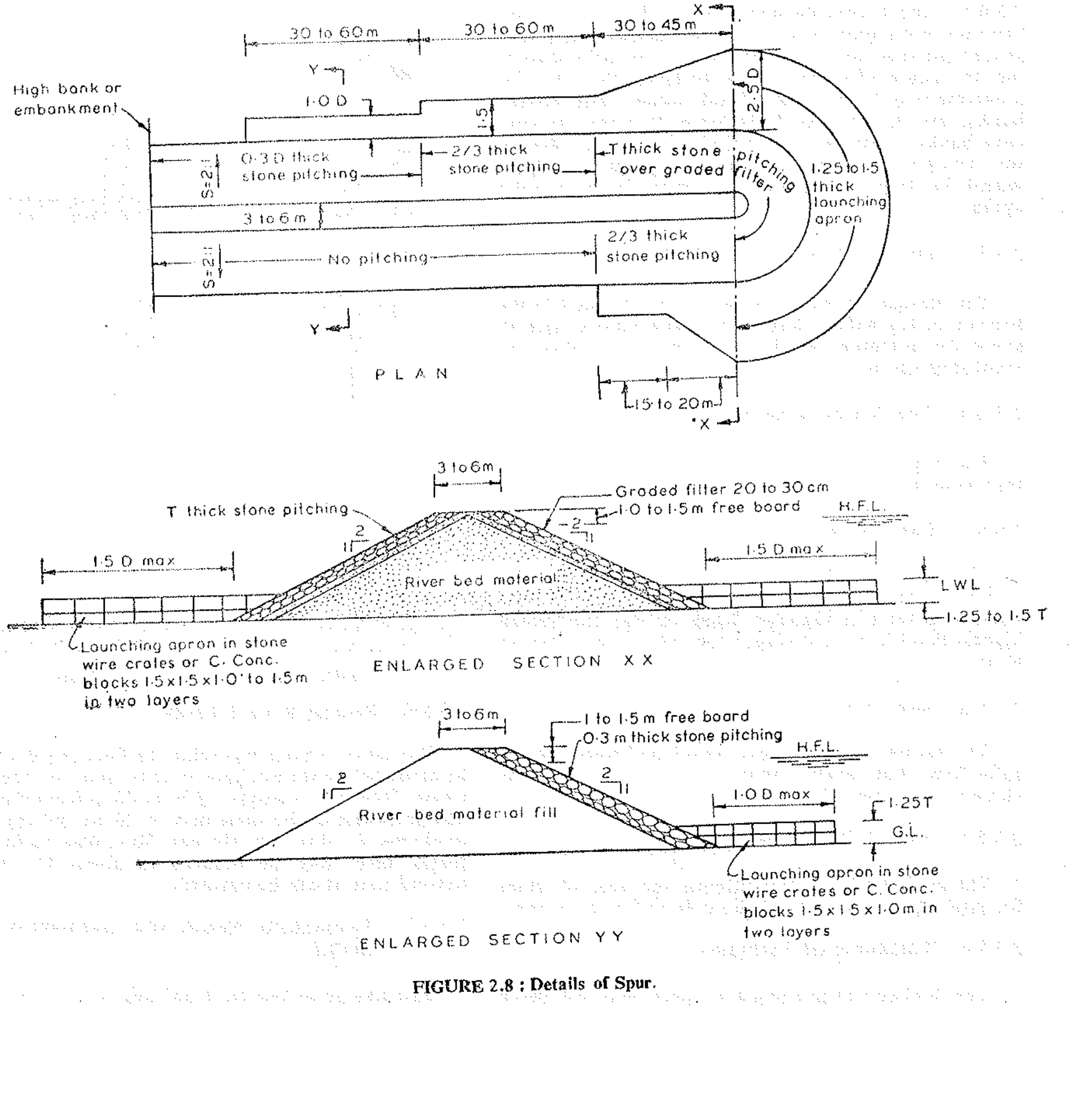


FIGURE 2.8 : Details of Spur.

TABLE 2.4.

Thickness of Pitching for Groynes or Spurs

Sl. No.	Reach	Thickness of Pitching
1	Semi circular nose and 30 to 4.5 m of upstream shank from	T
2.	Next 30 to 45 m of upstream shank	2/3 T
3.	Rest of the length of upstream shank up to the bank	0.3m
4.	30 to 60 m length beyond nose on the downstream shank	2/3 T
5.	Rest of the length of downstream shank up to the bank.	Nominal thickness of No pitching depending on flow conditions and material of construction.

TABLE 2.5.

Scour Depth for Spurs

Sl. No.	Reach of the Spur	Maximum Scour Depth
1.	Nose	2.0 D to 2.5 D
2.	Transition from Nose to shank and first 30 to 60 m in upstream	1.5 D
3.	Next 30 to 60m in upstream	1.0 D
4.	Transition from Nose to shank and first 15 to 30 m in downstream	1.0 D

The details of the various dimensions for spurs are depicted in Figure 2.8.

2.6. Cut-offs

Cut-offs as river training works are to be carefully planned and executed in meandering rivers. The cut-off is artificially induced with a pilot channel to divert the river from a curved flow which may be endangering valuable land or property or to straighten its approach to a work or for any other purpose. As the cut-off shortens the length of the river, it is likely to cause disturbance of regime upstream and downstream till readjustment is made. A pilot cut spreads out the period of readjustment and makes the process gradual. Model tests come in handy in finalising this form of river training works wherever needed.

2.7. Bank Pitching and Launching Apron

Though bank pitching has the limited function of protecting a given stretch of bank from erosion and since both bank pitching and river training works influence each other, it is treated as one of the training measures. The banks after the abutment and return walls are cut to stable slopes varying from 1 : 1 to 2 : 1 and then pitching is provided. Usually stones are used for pitching though local materials could also be

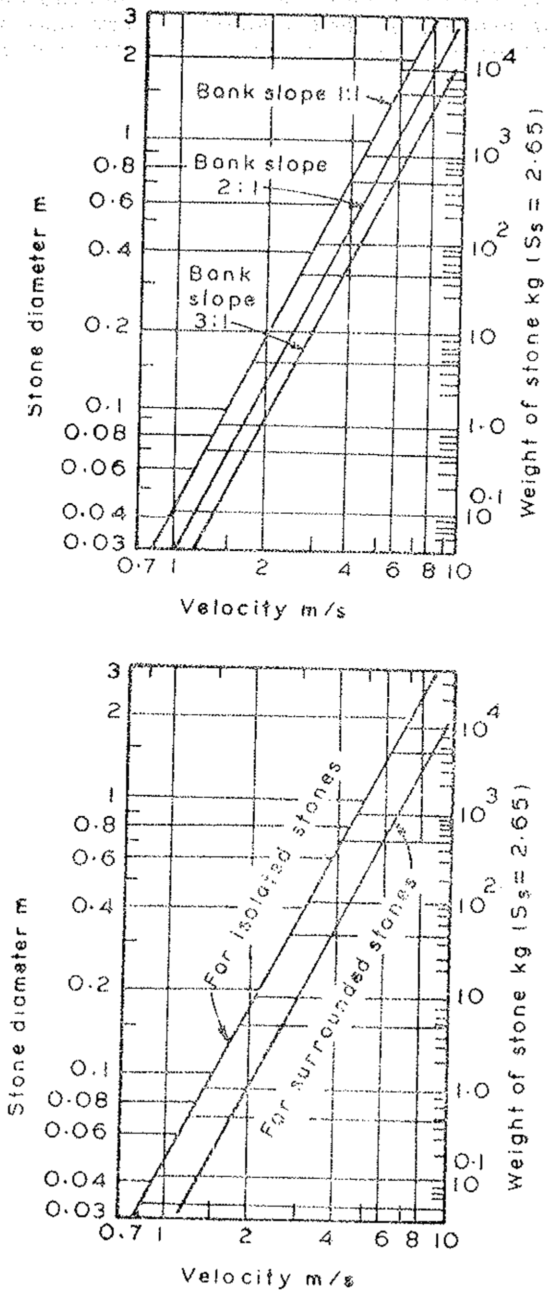


FIGURE 2 9 : Dimension of Pitching and Stone Aprons.

considered provided safety of slopes against erosion is ensured. The cement concrete block protection of the diversion structure both on the upstream and downstream beds can be continued on the bank slopes and then stone pitching beyond the C.C. Blocks.

2.8. Pitched Islands

A pitched island is an artificially created island in the river bed protected by revetment or stone pitching on all sides. On account of the turbulance created by the islands in their vicinity, the river channel is depended and the concentration of flow is increased in their direction. They may thus be utilised to attract the current towards themselves to reduce undue concentration on the opposite side or to direct and maintain the approach to some engineering work in the desired direction. Since experience on the use of pitched islands is rather limited, help of model tests must be utilised if such measures are to be adopted.

2.9. Sills

Sills, also called submerged sills or submerged dykes are used to counteract the tendency to excessive scour and large depths in a part of the river cross-section. This may happen at a sharp curve or adjacent to natural or artificial length of hard inerodible bank in a river otherwise consisting of easily erodible material.

CHAPTER 3

Fish Pass

3.0. Introduction

Fish Ladders or Fish Locks are components of Diversion structures wherever the problem of migratory fish exists. In large rivers certain species of fish always move from one reach to the other. They even travel from sea to a considerable distance along the river to the upstream and vice versa. The species of fish leave the cold water of the upstream reaches during winter and move down to relatively warm water in the plains. Before the start of monsoon, they move again upstream to clearer waters. Some fish-spend their adult life in the sea and migrate up the estuaries and fresh water rivers for spawning. When a diversion structure is constructed across the river, the passage of the fish is obstructed. Hence for the protection of fish, especially of migratory types, it would be necessary to construct suitable fish pass as a component of the diversion structure. Aspects relating to their location, investigation and data to be collected, type of fish-passes, design requirements, etc., are discussed in this Chapter.

3.1. Location

The fish pass should be provided near the deep channel through which they always move about. The enterance and exist of the fish-pass should be away from the range of heavy overfall of water from the diversion structure. The fish pass is usually located near the divide wall of the structure between the sluice bays and spillway bays since water would always be available near the divide wall. It can also be provided at one or both of the abutments, but the availability of water and the velocity of flow near the abutments and the regulators would have to be considered.

3.2. Investigations and Data Required

The fish-pass to be provided in the diversion structure has to be properly planned for the type of fish-pass to be adopted, location and size of openings, height ; etc. It should not be just a passage connecting upstream and downstream for the movement of the fish. The passage has to be smooth and there should not be any undue exertion needed for the fish to move about. Improperly designed fish-pass may not even attract the fish to effect the migration and sometimes may prove fatal also to the school of fish. For planning and designing an effective fish pass, certain data should be collected after thorough investigations and the data analysed. Important among them are given in the list below :

3.2.1. Type of fish, their characteristics, habits, instincts and environmental behaviour.

3.2.2. Place of spawning—whether at particular spots or anywhere.

3.2.3. If anadromous in nature, location map of the river/streams showing the migratory route and spawing grounds.

3.2.4. Timing of upstrean and downstream migration, nature of river flows, turbidity and temperature of water during these periods.

3.2.5. Maximum number of fish passing per hour during peak migration period.

3.2.6. Requirement of water per fish when confined to keep it in normal condition.

3.2.7. Nature of migration : Whether in groups or in line.

3.2.8. The number of fish passing upstream of the proposed structure for the previous periods say for a period of 30 years.

3.2.9. Commercail value of the fisheries they

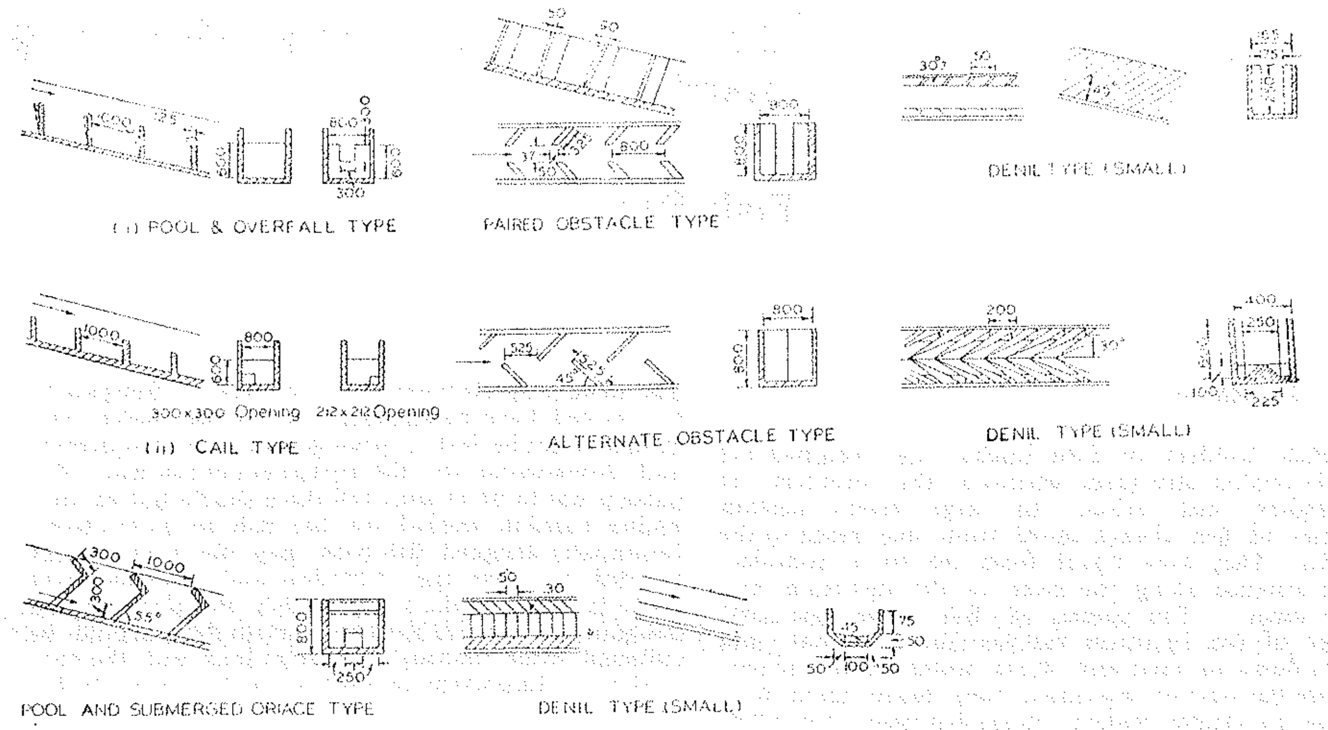


FIGURE 3.1 : Types of Fishways

contribute—to work out economics of having a fish pass or not.

3.2.10. The normal swimming speed of fish, its darting speed and preference for swimming or jumping

3.2.11. The stimulii of fish—Whether it is attracted by large volume of water or greater velocity.

3.2.12. The normal time that fish takes from downstream to spawning grounds and the delay which it is capable of withstanding without and detrimental effect on its biological functions.

3.2.13. The size of downstream migrants.

3.2.14. The nature of spawing grounds and the river condition at the time of spawning, viz, velocity, depth, turbidity and temperature of water.

3.3. Types of Fish-pass

The fishpass can be of two types viz., fish ladders and fish locks. The fish ladders can be further subdivided in to four types as (*i*) Pool and Jet fishway and (*ii*) Paired obstacle fishway (*iii*) Alternate obstacle fishway and (*iv*) Denil fishway. A description of these are given below :

3.3.1. *Fish-Lock*

The operation of fish lock is similar to that of navigation lock. A group of fish is admitted in to the lock at a time and transferred either to upstream or downstream as the case may be. The lock is generally located next to the divide wall.

3.3.2. *Fish Ladders*

A fish ladder usually consists of a series of pools with small drops in water surface at the cross walls. Drops in water surface are generally about 0.3 metre, but could be as great as 0.6m at short ladders where only a few jumps are necessary. Long ladders are often provided with intermediate rest pools. The migratory fish jump from one step to the other. The different types of ladders are briefly described below :

3.3.2.1. Pool and Jet Fishway

The pool and Jet fishway consist of a series of pools in a stepped arrangement from tail water level to the upstream water level. The pools may also be connected by short sloping channels if the fishway is cut in rock or they can be formed by a series of obstacles or dams placed in a sloping channel. In the latter case, the flow from pool to pool may be over solid obstacles or through notches or orifices in the obstacles or a combination of these.

Each pool affords a more or less adequate resting place for the ascending fish according to the size and design of the pools. When the obstacles have submerged orifices, the fish generally prefer to use them thus avoiding exposure, In negotiating an ordinary overfall the fish usually swim up the falling jet. The tendency to jump from pool to pool is limited to certain species of fish. In the case of an overfall, the fish must raise itself from one pool elevation to the next and in the case of an orifice the fish must work against the two sides of orifice.

The Pool and Jet fishway in its many variations is probably the oldest and most widely used of all types There are three sub-types of the Pool and Jet fishway, viz. (*i*) Pool and Overfall Jet type (*ii*) Cail type and (*iii*) Pool and Submerged orifice type. These three types have been studied by the Iowa Institute of Hydraulic Research and some aspects of the same are given in Appendix 3.1.

These types are shown in Figure 3.1.

3.3.2.2. Paired Obstacles Fisway

The paired obstacles fishway consists of a sloping pairs spaced along the channel at intervals equal to about the width of the channel and a straight free passage between them. The free passage is considerably restricted and one half to five eights of the channel width being taken up by the obstacles. The floor in the free passage is usually flat, although small bottom obstacles may be used. The wide spacing of the pair of obstacles along the channel gives the fishway the hydraulic function of a pool and jet type fishway. but since the pools are divided by the central flow, they do not afford much of a resting place except to the smaller fish. When the Denil type is compared with the paired obstacles fishway, the ratio of preference is 5 : 1 in favour of Denil type. Cat-fish does not prefer the Paired Obstacles Fishway.

This type of fishway is shown in Figure 3.1.

3.3.2.3. Alternate Obstacle Fishway

Alternate obstacle Fishway consists in general of a straight rectangular channel with obstacles or baffles placed alternatively along the sides producing a jet deflection in the horizontal plane. This type of fishway has been built in an almost unlimited variety of baffle shapes, spacings and angles. The flow is confined to zig-zag path which is much longer than the fishway. The width of passage is effectively less than its measured width because of its tortuous form. Velocities are sharply localised and are often in such a direction as to strike the fish from the side. The water depth is usually irregular and there are many whirl pools or vortices. There is no adequate resting space for the fish. Baffle arrangements can be made which minimise these difficulties and provide a fair passage for fish, if the slopes used are mild. This type of fish-way is not quite satisfactory. A type of this fish-way with a reasonable slope of say 1 in 4 is shown in Figure 3.1. When compared with Denil type, equal number of fish use both these two types.

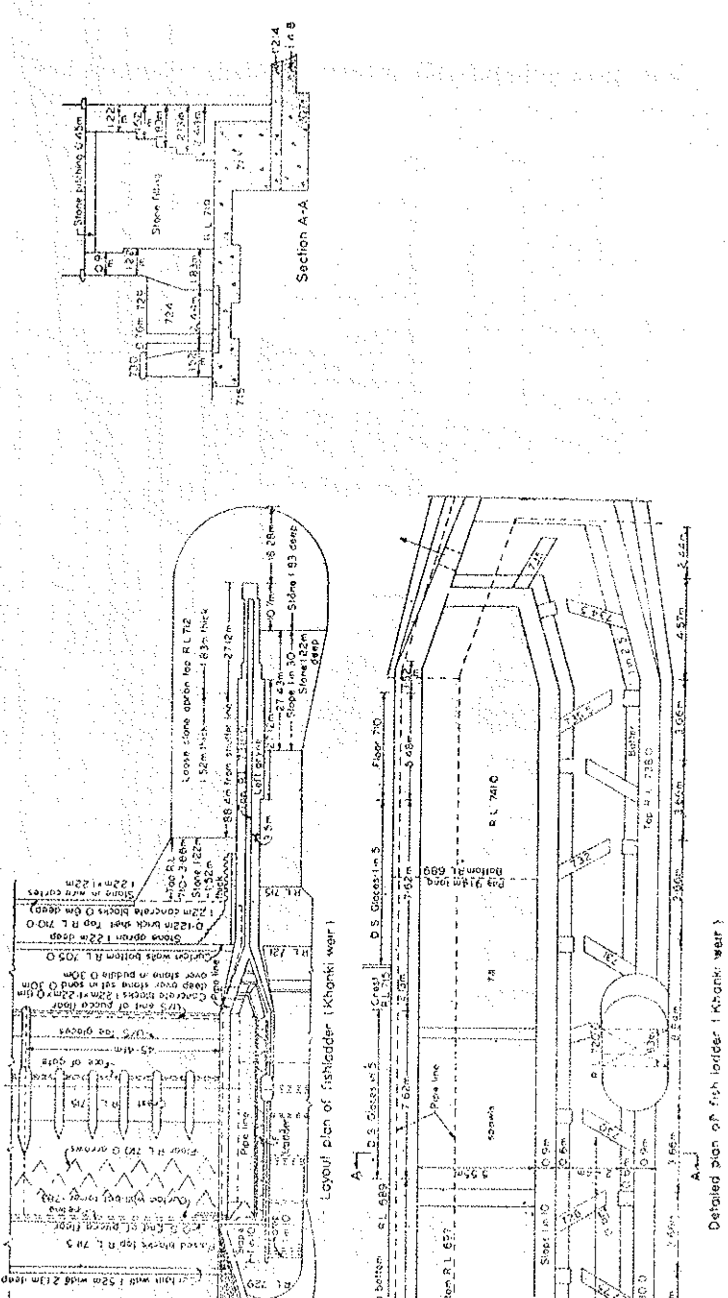


FIGURE 3.2 Fish Ladders at Khanki Weir

3.4.5. Typical details of Fish ladders in Khanki weir are shown in Figure 3.2. Typical details of Fish ladder provided in Kosi Barrage, Bihar are shown in Figures 3.3 and 3.4.

3.4.6. Typical details of fish lock are shown in Figure 3.5.

3.5 Design Requirements

The successful function of a fish ladder primarily depends on its location to attract the migratory fish. A number of factors such as physical features of the location, the height to be overcome and the available water supply may influence the design of a fishway. A large fishway with abundant flow of water may prove advantageous as the outfall would provide a greater attraction to the fish than that from a small one. However, some times, conditions may limit both the dimensions and available quantity of water. The different types would need the design of a graded section of such length and dimensions as would provide a series of compartments through which water would pass to facilitate the fish to move upstream and downstream. The dimensions of the fishway, supply of water and grade would have to be suitably adjusted to reduce the turbulence in the compartments to the point where the fish variety would not become fatigued. Help of hydraulic model tests should be taken wherever necessary.

3.5.1. The following points may serve as guidelines in a satisfactory design of a fishway.

3.5.1.1. The entrance to the fishway should be from a poll in which the fish would naturally collect when their further progress towards the upstream is prevented by the construction of the diversion structure.

3.5.1.2. The entrance should be well submerged at all stages of water when the fish are seeking ascent through it.

3.5.1.3. The size of the free cross-section of the fishway should be sufficient for the unhampered swimming movements of the fish with due consideration for their sizes and number using the fishway. The width should not be too narrow.

3.5.1.4. The hydraulic conditions created should be such that the passage of fish up the fishway does not overtax the energy of the fish. No large scale vortex or whirl poll effect should be present.

3.5.1.5. The downstream openings should be self-attracting.

3.5.1.6. During the periods of migration, water should be available in the ladders.

3.5.1.7. Drops in water surface between successive polls are generally limited to about 0.3 m., but could be as great as 0.6 m, at short ladders where only a few jumps are required.

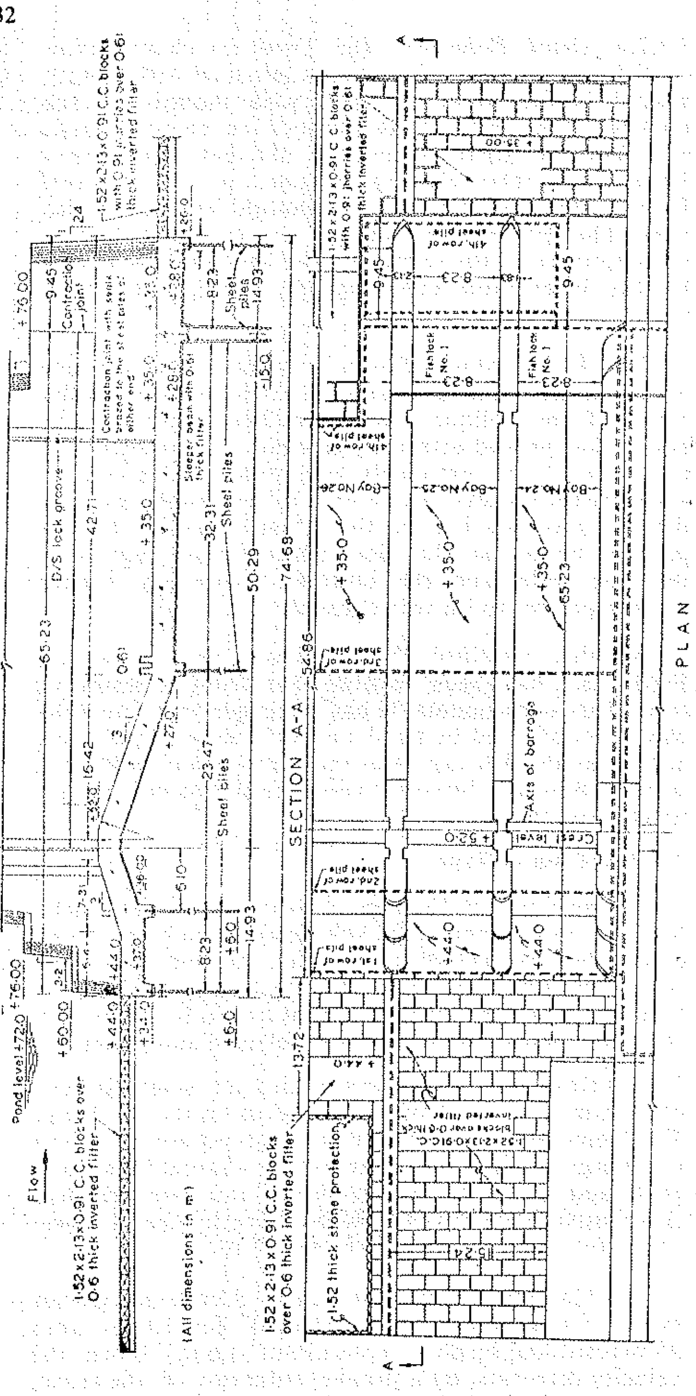


FIGURE 3.5 Typical Details of Fish Lock

3.5.1.8. Longitudinal slopes may vary from 8 : 1 for low lifts to 15 : 1 for high lifts.

3.5.1.9. Flows of water through fish ladders should be fast enough to lure the fish. Velocities of about 0.5 m/sec are generally sufficient. The velocity through ladders should not exceed 2.5 m/sec.

3.5.1.10. Poll depths usually vary from about 1 m., at small ladders to about 2 m., at large ladders.

3.5.1.11. At weirs on small streams, pools may be from 1.25 m to 2.5 m wide and 2 m to 3 m long. At weirs on large rivers, pools may be 6 m to 12 m wide, but never more than 6 m long.

3.5.12. At weirs where water supplies are ample, entire lengths of cross-walls between pools can act as weirs and where the supplies are limited, notches provided in the cross-wall between pools can act as weirs. These notches in the cross-walls can be provided either at centre or staggered at the ends. Holes or openings in the cross walls are preferable as they permit passage of fish without jumping. The walls are generally built oblique and are staggered so that the fish can take rest after passing one hole before they move to the other.

3.6. Regulation

The upper end of the fishway should be provided with regulating arrangements so that the quantity o fish admitted can be controlled. Otherwise, the compartments would be unduly flooded during high water periods and insufficient supplies during low water periods. The best results are obtained by a series of regulating gates between compartments and the amount of water entering the fishway may be adjusted to suit the level above the structure.

3.6.1. Automatic regulation may be provided and is employed in some instances where the variation of water level is not too great by the provision of a series of compartments with appertures through the bottom of the dividing partitions, the dimensions of the appertures being reduced in each successive partition commencing at the inlet. The volume of water entering a fishway may also be controlled by a movable slit operated by floats.

APPENDIX—3.1

Experimental Observations on Pool and Jet Fishways

Investigations were carried out at the Iowa Institute of hydraulic Research where various designs of fishways using full scale models and different varieties of fish were compared. The experimental observations on the three sub-types of Pool and Jet Fishways are given below :

A. **Pool and Overall Jet :** Closer spacing of baffles was not recommended. With narrow range of 100 to 150 mm depth over baffies, the appearance of the flow was good. The energy from each overfall was dissipated in the succeeding pools. The baffles were first installed vertically, but it was observed that the fish have difficulty in passing over the square edge of the baffle. Due to this difficulty and also for the purpose of increasing the circulatory motion in the pools, the baffles were tilted upstream by 125 mm. This would also help the fish to rest in the pools The attraction at the foot of the fishway was particularly good in this arrangement.

This fishway was compared with the Denil type and also with the submerged orifice fishway. The fish using the submerged orifice fishway out numbered those using overfall fishway by more than 4 to 1 whereas those using the Denil type out numbered it almost by 8 to 1.

With notched obstacles, the results were found to be the best when the notch flowed full and when there was no flow over the top of the baffle. The notching of the obstacle tended to cut down the jump height and also the fish were less exposed in passing an obstacle. The attraction to the fish wasfairly good and the water required was less. It was less sensitive to head water changes than the straight overfall. When this fishway was compared with the Denil type, the preference to Denil type was about 7 to 1.

B. **Cail Type** : The pools were about square in order to produce the desired circulatory motion and all flow should be through the orifices. The circulation in the pools was moderate and the resting facilities were excellent. Fish were able to ascend this fishway even when the flow was low enough to expose the orifices, but it was advisable to maintain water up to the baffle depth. When compared with Denil type, the preference to Denil type varied from 2. 5 : 1 to 4.5 : 1.

C. **Pool and Submerged Orifice Type** : Ordinary Pool and Jet fishways with staggered orifice were usually low in efficiency, The jet deflection was moderate and hence the energy dissipation was also moderate. However, a design based upon strong jet deflection in the vertical plane was characterised by cross-walls slanted strongly downstream with submerged orifices in arial disposition. The minimum depth for effective operation was determined bydepth necessary for swimming depending on size of the fish rather than the mechanical effect. The tail water when submerged had no attraction other than the underwater current. To increase the attraction, a medium flow with the low jet partially exposed was to be maintained. The medium to low range was better than the medium to high flow range. This type of fishway was compared with the straight overfall type. The ratio of preference was more than 4 to 1. When compared with the Denil type under average conditions, the total number of fish using either of them was the same.

CHAPTER 4

Navigation Facilities

4.0 Introduction

With increasing demand for provision of navigation facilities wherever possible, it is necessary to make a comprehensive planning of the diversion structures including provision of navigation locks. It may be necessary to allow traffic through the river from upstream to downstream and vice versa or from upstream or downstream to a feeder canal/link canal and vice versa. The various types of navigation locks which are necessary for this purpose, other facilities to be provided, their layout and design considerations, etc., are all discussed briefly in this Chapter.

4.1. Data Required to be Collected

For proper planning of the navigation facilities to be provided, it is necessary to collect certain data. These are given below :

4.1.1. *Topographical Data*

The data collected for planning of the diversion structure and the canal system can be made use of.

4.1.2. *Hydrological Data*

Hydrological data collected for the diversion structure like maximum and minimum flows, water levels, etc., can be made use of.

4.1.3. *Sub-surface Investigations*

Geological data including foundation conditions, foundation and leakage problems, etc., at sites suitable for location of navigation locks in the vicinity of the diversion structure should be collected.

4.1.4. *Navigation Requirements*

For the navigation facilities to develop their maximum usefulness and work efficiently, it is necessary to collect data on the frequency and density of traffic, visibility, ease of approach, lockage time admissible, hydraulic flow conditions at the entrance and exit, existence of bridges nearby, adequate mooring facilities and maneuvering areas, etc. These would help in the determination of the size of the locks and other facilities required.

4.2. Types of Navigation Lock

Navigation lock can be of different types. Stone masonry construction has been replaced now a days by concrete structures due to better resistance to impact, abrasion and deterioration. The various types of navigation locks are constructed of gravity or mass concrete, dry-dock reinforced concrete, steel sheet pilling, reinforced concrete and combination type.

4.2.1. *Gravity or Mass Concrete Type*

This type of lock is composed of wall sections which resist applied loadings by their massiveness. Size and shape of sections should be selected to fit the particular purpose and loading condition, sliding and foundation stresses. Top and intermediate widths are made to provide space for such installations as filling and emptying systems, anchorages for movable structures, operating equipment, temporary closure structures, their mechanisms and miscellaneous accessories.

4.2.2. *Dry Dock Reinforced Concrete Type*

The dry-dock type lock consists of relatively thin walls constructed integrally with a thick floor slab. They are designed to act together as mono , each being heavily reinforced to distribute the loads.

4.2.3. *Reinforced Concrete Type*

This is the usual type now a days adopted. The walls may be either cantilevered or counterforted.

4.3. Components of Lock Walls

Navigation Lock-walls are sub-divided into various parts and are designated in regard to their position or purpose such as lock chamber walls, upper gate-bay walls, lower gate-bay walls culvert in take walls, culvert discharge walls, upper approach walls and lower approach walls.

4.3.1. *Lock Chamber Walls* :

The lock chamber walls inclose the lock between the upper and lower gate bays. These are further divided into monoliths provided in between with seals.

4.3.2. *Upper and Lower Gate-bay Walls* :

The gate-bay walls include those portions of the lock in which the gate recesses, gate anchorages, gate

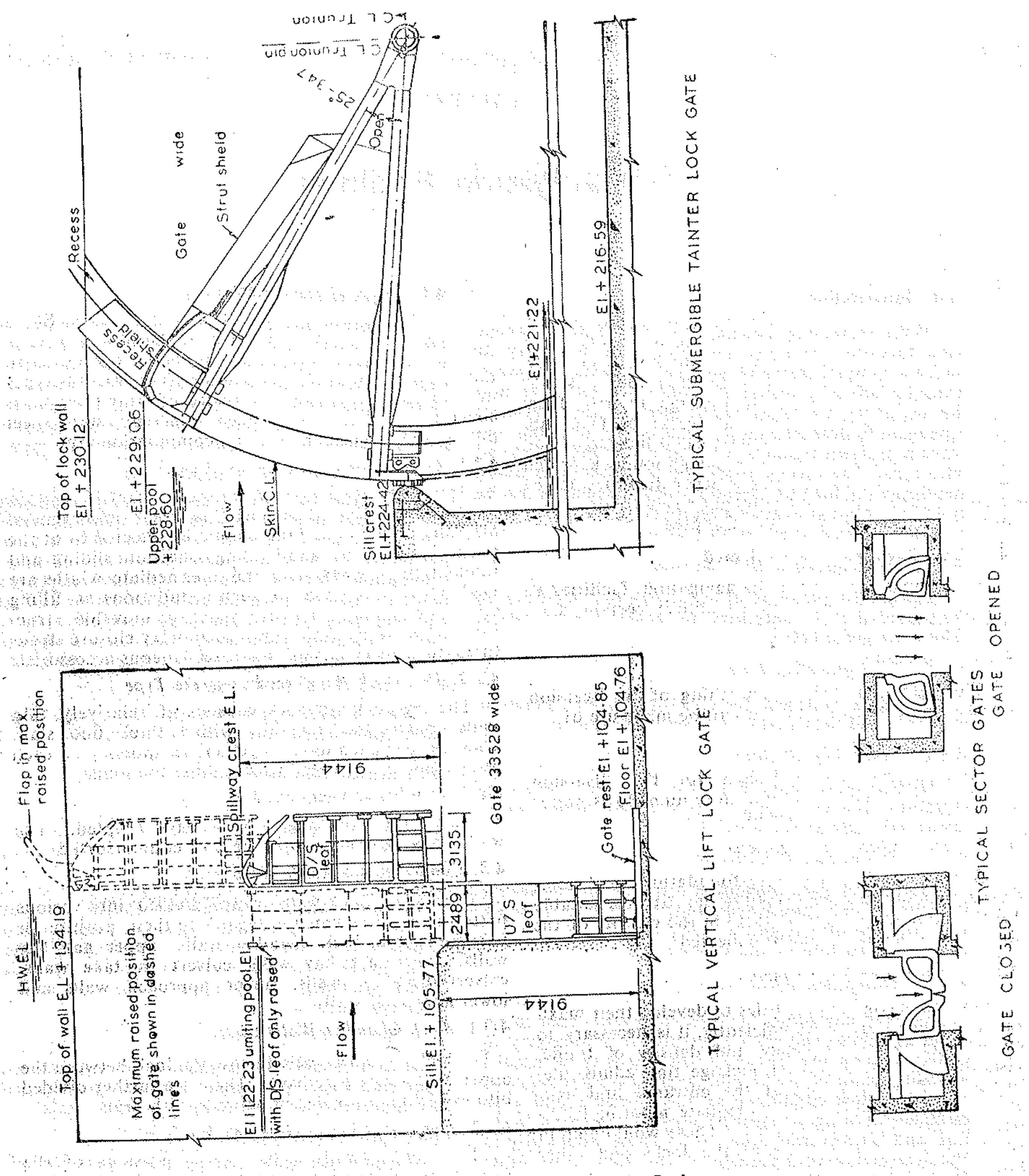


FIGURE 4.1 : Types of Gotes for Navigation Lyck.

machinery and sometimes culvert valves and culvert bulkheads are located.

4.3.3. *Culvert In-take and Discharge Walls*

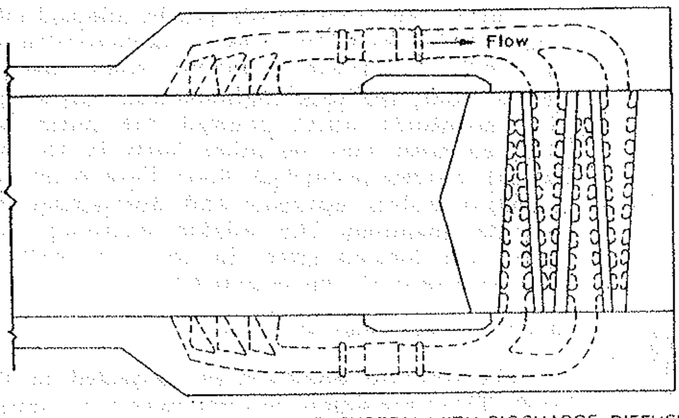
In take walls are those extended immediately beyond the upper gate bays to provide space within which to form intake ports leading to the culverts. Discharge walls are the extensions from the downstream end of the lower—gate monoliths exclusive of the approach walls.

4.3.4. *Approach Walls*

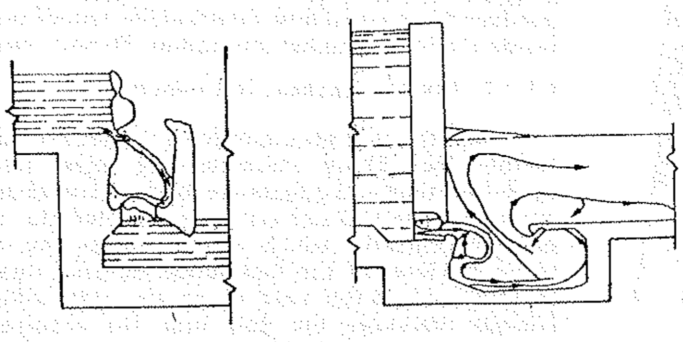
Provision of approach walls at each end of the lock facilitates lockages by reduction of hazards and increases the ease of entrances and departures of craft. They also provide means for checking the progress of a tow or correcting its alignment by putting out lines to check posts on the wall, thereby avoiding extremely slow, tedious approaches. They provide mooring spaces for the separated part of tows which are too long to negotiate the lock in one lockage and thus aid in the safety and speed of lockage.

4.3.5. *Lock Sills :*

Lock sills form the fixed portion of the damming surface under the service gates or temporary closures. Gate sills are utilised for each lockage. At or below the



STUB CULVERT HYDRAULIC SYSTEM WITH DISCHARGE DIFFUSER



FILLING SYSTEM EMPTYING SYSTEM

FIGURE 4.2 : Typical Filling & Emptying Systems of Navigation Lock.

gate sills, emergency closure sills are provided to aid closure structures seal at the bottom and have a base for support. The closure structures at each end of the lock are sometimes considered necessary to stop the flow through the lock chamber if the gates become inoperative. They are also required to permit unwatering of the lock chamber for periodic inspections and repairs.

4.3.6. *Lock Floors*

Lock floors are required to prevent erosion of the base.

4.4. Lock Gates

Lock gates are provided at the ends of a lock. Various types of gates are in use such as mitre gates, rolling gates, sector gates, tainter gates and lift gates, the common ones being mitre, tainter and lift gates.

4.4.1. *Vertical Lift and Tainter Lock Gates*

Vertical lift gates can either be submerged below the level of the upper sills or raised out of the locks permitting the traffic to pass through. Typical submergible vertical lift lock gate is shown in Figure 4.1 and submergible tainter gate is also shown in the same figure. In the open position the gate is submerged immediately downstream from the gate sill. When functioning as a filling device, the tainter gate is gradually submerged below upper pool level and flow over the crest drops into the lock chamber in the manner of a spillway or weir. Submergible gates are practical only where the sill is sufficiently high to permit the gate to be dropped completely below its top surface. Submergence into a floor recess is not considered advisable as the recess would get filled up with silt and debris.

4.4.2. *Sector Gates*

In tidal reaches of rivers or canals locks may be subjected to reversal of heads. The gates should be capable of withstanding hydrostatic load from either side. Sector gates (shown in Figure 4 2) serve the dual purpose of service gates and device for filling and emptying the lock.

4.5. Filling and Emptying Systems

Successful and smooth handling of the vessels through the lock depends on the efficient and effective operation of the filling and emptying systems. As any inefficient filling and emptying system produces unfavourable velocities or surges in the lock chamber adding to the difficulties of tedious and time consuming lockage operations, the system has to be well planned and designed.

There are two general classes of types of filling and emptying systems, viz., (*i*) end filling or emptying systems which may utilise valves mounted in the lock service gates or short valved culverts located in the lock walls adjacent to the service gates or may provide for flow either through, over or under partially open lock service gates and (*ii*) systems that utilise longitudinal culverts in lock walls or floors with multiple ports into the lock chamber to provide a uniform distribution of flow. In the case of a large lock, any one or both the systems can be adopted.

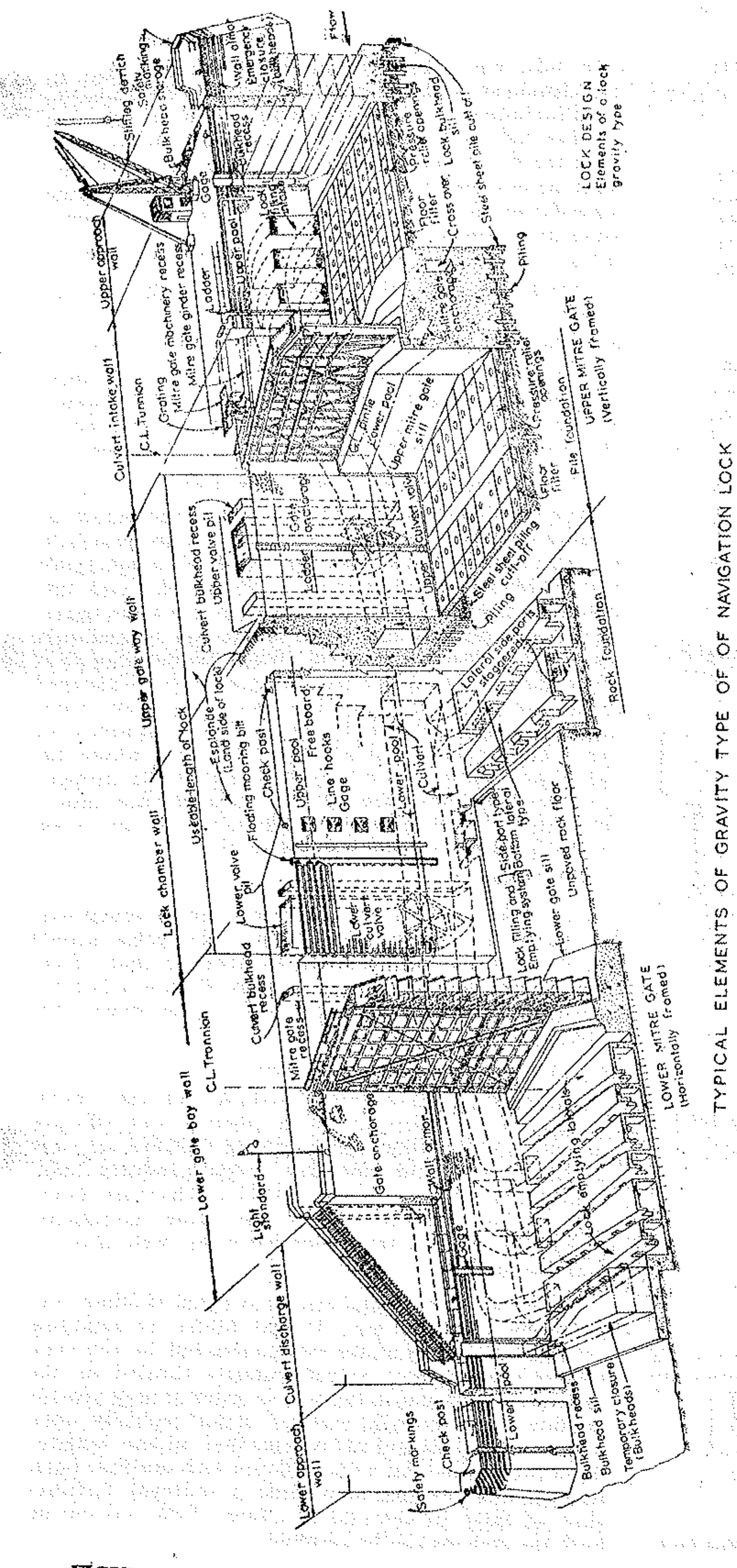


FIGURE 4.3, Typical Elements of Gravity Type of Navigation Lock,

4.5.1. *Valves in Service Gates or Gate Sills*

In this system of filling and emptying system, one or more valves of the slide gate or butterfly type are installed in the leaves of the service gates of the lock. The valves can be operated manually or mechanically. These ares suitable only for low-lift locks with relatively small chambers where fast rates of filling are not required. In this system, deflectors or baffles downstream of the sill to dissipate the jet action and turbulence created by the inflowing water are provided.

4.5.2. *Stub or Loop Culverts*

Sometimes, stub or loop culverts, located in the lock walls adjacent to the service gates are used for filling and emptying. The upstream legs of the culverts are connected to intake ports or manifolds located in the lock walls, gate sill or floor of the lock or lock approach and faced toward the upper pool. The downstream legs terminate in similar ports or manifolds immediately below the upper service gate. Similar culverts controlled by valves are provided for emptying into the downstream pool. Diffusers are usually required to dissipate the turbule nce created by the discharge from the stub culverts. These types of culverts can be used when continuous culverts in the lock walls are not practicable to be constructed.

4.5.3. *Side Wall Culverts*

This system is one of the usually adopted filling and emptying systems. It consists of continuous culverts in each lock wall with manifold connections to the upper pool, the lock chamber and the lower pool. The manifolds which connect the culvert with the lock chamber may be either ports in the walls or lateral culverts in the lock floor. Flow is controlled by means of valves upstream and downstream from the chamber manifold. The intakes consisting of multiple ports are located either in the side walls of the approach or in the upper gate sills.

4.5.3.1. WALL PORT MANIFOLDS

The wall port manifolds are provided in the lock wall. These are square or rectangular in section and their entrance are streamlined. They are also located staggered in the opposite walls. These arrangements produce more equal flow distribution, reduce turbulence inside the lock chamber and reduce Hawser stresses.

4.5.3 2. BOTTOM LATERAL MANIFOLDS

When the wall-port manifolds do not provide satisfactory lock filling characteristics with respect to operation time and quiescence, bottom lateral manifolds are used. The wall culverts are connected to a series of lateral culverts which extend across the chamber below the level of the lock floor. Several small ports located either in the roof or in the side walls of the laterals discharge the flow into the chamber. Since bottom lateral manifolds are more expensive, economics would have to be worked out with reference to savings in operating costs and satisfactory filling and emptying conditions.

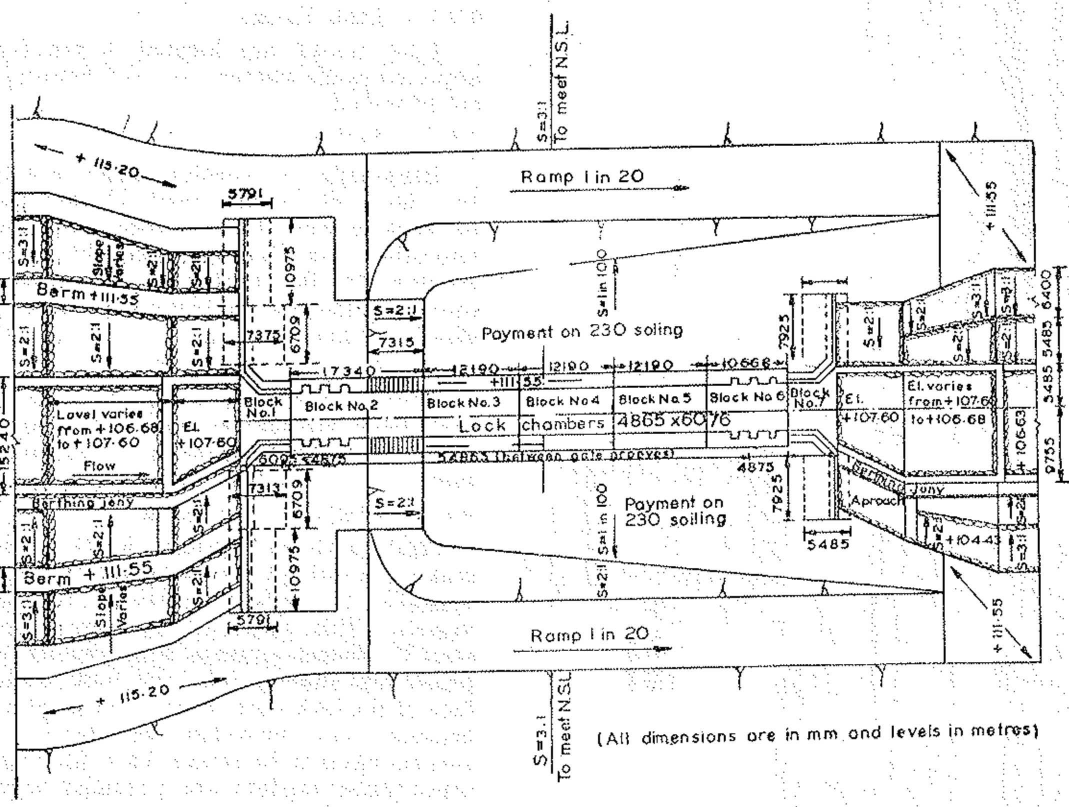


FIGURE 4.4 : Typical Detailed Plan of Navigation Lock.

4.5.4. *Longitudinal Culverts in Lock Floor*

If it is not possible to provide culverts in the lock walls due to foundation conditions, available space at the project site etc., longitudinal distribution of the discharge in filling operations can then be obtained by placing the supply culverts in the floor of the lock parallel to the side walls of the lock.

4.5.5. *Outside Intakes and Discharge Systems*

If the lock is rather long and lift is high, to reduce the filling and emptying times, sometimes it would be advantageous to draw or discharge water from outside of the upper or lower approach say the pond or river itself. Such arrangements are termed outside intakes and discharge systems.

4.5.6. *Auxiliary Emptying Systems*

As the co-efficient of discharge for emptying is usually lower than that for filling in the case of a side wall culvert, emptying operation may take more time. For speeding up the opera- tion, auxiliary emptying system can be provided in the form of a stub culvert.

4.6. Miscellaneous Provisions

There are a number of miscellaneous provisions to be made in a lock for efficient functioning, safety and inspection. Some of them are Towing and Snubbing facilities, Esplanade, Protective Equipment, Wall Face Protection, Guard Rail and Parapets, Ladders, Stairs and Ramps and Ganges and Recording Devices.

4.6.1. *Towing and Snubbing Facilities*

The Towing and Snubbing facilities include equipment for towing and snubbing, snubbing lines, floating mooring bits and line hooks.

4.6.1.1. Equipment for Towing and Snubbing

When the length of a tow is greater than that of the lock, it is divided into two or more sections while the part with its own power can pass without any aid, others may have to be moved through the lock and snubbed beyond.

4.6.1.2. Snubbing Lines

Snubbing lines are provided to afford some safety against the possibility of the gates being rammed by the vessels or tows passing through the lock.

4.6.1.3. Floating Mooring Bitts

Floating mooring bitts are provided to avoid the necessity of changing lines as the pool varies. Their spacing would vary according to the length of lock and the type of tows which normally use the waterway.

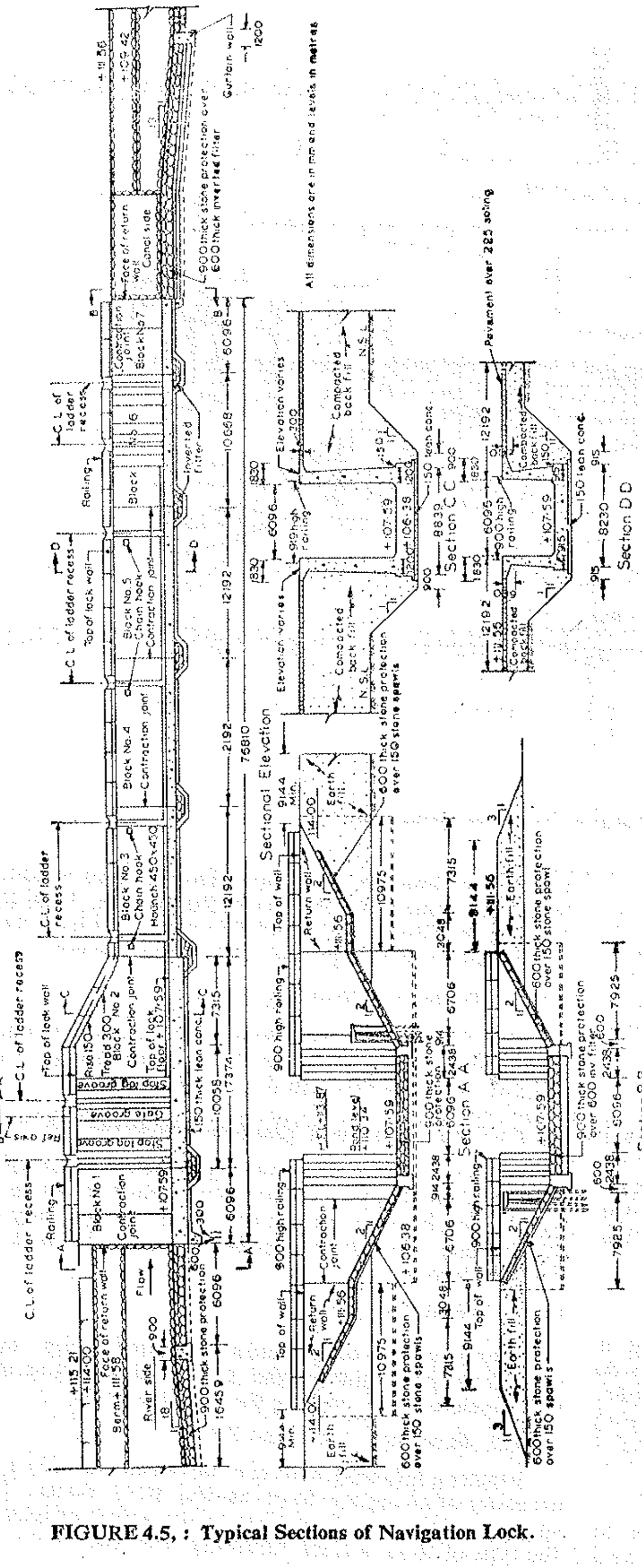


FIGURE 4.5, : Typical Sections of Navigation Lock.

Floating mooring bitts should never be allowed to sink.

4.6.1.4. LINE HOOKS

Line hooks are located in the face of lock and approach walls whether or not floating mooring bitts are provided.

4.6.2. *Esplanade*

Esplanade or working area is usually provided on the back fill or the earth closure section widened to requirements for storage, maintenance, parking and other miscellaneous uses. While determining the features of the esplanade, access to lock, lighting, protective fencing and other miscellaneous requirements should be considered.

4.6.3. *Protective Equipment*

Protective fenders are provided to protect the lock gates from damage due to impact from a vessel passing through the lock chamber. This can be of wire rope type.

4.6.4. *Wall Face Protection*

Wall surfaces against which vessels rub when making a lockage are protected by a smooth surfaced material to reduce abrasion on both walls and vessels. This could be by rows of rolled structural steel T - shaped sections with slightly convex surface projecting slightly into the lock chamber from the face of the lock wall. Protection angles with rounded corners are provided on vertical or horizontal corners likely to be struck by a moving vessel. Sometimes timber fenders are provided horizontally along the approaches.

4.6.5. *Guardrail and Parapets*

When the backfill is a sufficient distance below the top of lock and approach walls, guard rails are provided. Other facilities such as stair wells, ladder recesses and other openings at the top of walls and at other locations not protected with covers should have guard-rail or other protective equipment on all sides.

4.6.6. *Ladders, Stairs and Ramps*

Ladders should be provided at suitable intervals to provide access to the lock floor, floating accessories and other components for carrying out necessary maintenance and safety measures. Wherever less than three risers are involved, ramps could be provided instead of stairs.

4.6.7. *Gauges and Recording Devices* :

Gauges and Recording devices are incoporated in the lock walls at suitable locations to indicate the water levels.

4.6.8. The different elements of a navigation lock are shown in Figure 4.3.

4.7. Hydraulic Designs

After a suitable location for the navigation lock is fixed, hydraulic designs should be done to satisfy the basic requirements including (*i*) A dependable depth for movement of crafts in the navigation channel and lock. (*ii*) adequate widths in the channel and lock for maneuvering crafts at the desired speeds (*iii*) freedom from hazardous currents and (*iv*) a minimum time requirement in the passage of locks. The hydraulic designs are done generally to ensure adequacy of water supply for lockages, fix top levels of the components of the structure, provide approach structures, provide efficient filling and emptying systems and to fix other necessary features. It is always desirable to conduct hydraulic model tests to accomplish lockages with reasonable time and avoid excessive turbulence and Hawser stresses.

4.7.1. *Water Requirements* : It should be ensured that navigation pools at their normal levels are always maintained. The various factors such as water requirements for lockages, leakage, seepage, evaporation, etc., should be considered. The flood value beyond which navigation is not to be permitted should also be determined.

4.7.2. *Elevation of Lock Walls* : The top of the lock walls should be carefully fixed so that the lock is useable at all times except during large floods. The lock walls should provide protection to the adjoining areas. Usually 2.25 m or more of free board above the normal pool level would be required to obtain satisfactory operating conditions. The portions upstream and downstream of the gates should have top elevations above the corresponding high flood levels with sufficient free board.

4.7.3. *Lock Dimensions* : The size of the lock depends on the size of the largest boat it is designed to accommodate. There are clearances required to allow the vessel to enter the lock safely. The minimum clearances required are usually as follows :

Each side 0 75 to 1 m ; Between nose/end of boat to gate 1 to 2.5 m ; Length between gates = Boat length + 2 to 5 m ; Drought during Min. WL. 1 to 1.5 m ; Overall dimensions of lock are to be fixed suitably taking requirements for gates matters, etc.

4.7.4. *Lock Approach Structures* : The upstream and downstream approach structures should be designed after a careful study of the hydraulic characteristics of the waterway in their vicinity. These should ensure smooth entrance and exit of the crafts and reduce hazards caused by adverse currents.

4.7.5. *Filling and Emptying Systems* : Design of filling and emptying systems are very important in the proper functioning of navigation locks. General requirements to be satisfied in the designs are that (*i*) filling and emptying operations should be performed as rapidly as practicable, (*ii*) disturbances caused during the operation by the flow should not endanger any craft in the lock chamber or approaches and (*iii*) the filling and emptying system should be as economical in construction and operation as possible without sacrificing efficiency and safety. The disturbances due to flow during the lock operations could be either or both of (*a*) localised turbulence or rotational flow generated by jets of water coming into the lock chamber or lower approach and (*b*) an oscillatory longitudinal surging in the lock chamber. Of these two the second disturbance (*b*) is far more serious since both the craft and the structure could be damaged. Lengthening of the lock chamber and tows increase the potentialities of damage and hence surging should be reduced to a minimum

4.7.5.1 The requirement for speed in filling and emptying and the requirements for safety of traffic and structures should be given parallel consideration. Speeding up lockage times may lead to intolerable turbulence and surging in the lock chambers. Hence a suitable system and timings would have to be determined and model experiments come in handy for this purpose.

4.7.5.2 Surging causes the crafts in the lock chamber to drift from one end to the other end in order to keep them from stricking the gates or damaging other parts of the structure, it should be restrained by Hawsers. The stress in the Hawsers is essentially a function of the gross tonnage of the tow and the slope of the water surface in the lock with the frictional forces exerted upon the craft by flow in the chamber having only minor effects. Hawsers required to restrain the craft must be provided.

4.7.5.3. During filling and emptying operations, as the active head and discharge vary from maximum to zero, analysis becomes a little difficult. However, analytical methods based on empirical data have been developed and could be used to determine the size of the culverts and also end filling systems. Head losses in the intake and discharge manifolds, friction in the culverts and losses due to valves should also be taken into account in the analysis.

4.7.5.4. One of the initial steps in the design of filling and emptying system is the selection of tentative filling and emptying times. It is usually based on a study of other locks and hydraulic model test results. Roughly it is given by the formula $T=m\sqrt{\frac{H}{gz}}$ where H is the lift in feet, g acceleration due to gravity in ft/sec/sec and m is a constant varying between 600 and 1000, but generally between 720 and 960. An average value of m is 840.

Similar expressions for approximately determining the lock operation time are also available. For details, manuals on Navigation locks may be referred. These timings would later on be used to determine the size of the culverts, intake and discharge manifolds.

4.8. Structural Designs

The lock walls are usually provided as monoliths and joints are provided between them. They would be located according to the requirements of the particular element that the monolith supports or contains. Each monolith should be completely stable without depending upon adjacent units for support. The length of a monolith is generally kept less than 15 metres. The length of monoliths containing the gate anchorage, valves, filling and emptying system components and other machinery may, however, be more.

4.8.1. Complete stability analyses should be made at all horizontal planes of the structure where either the applied loads or the section changes abruptly and the stress conditions should be checked at points where other planes of weakness may be anticipated. Usually analysis is done on the basis of unit length of wall. However, the monoliths subjected to gate loads, line pulls, vessel impact etc., and loaded in more than one axis, are analysed as a whole. The stability of the lock walls would need to be checked for three different conditions, namely (*i*) Normal operating condition taking into account the worst combination of loads during a complete lockage cycle, (*ii*) Extreme operating, maintenance and emergency conditions taking into account unusual loads like vessel impact, extreme saturation levels, drawdowns, earthquakes, etc. Safety factors and stresses could be relaxed in this condition and (*iii*) Construction condition taking into account earth pressures with or without uplift and surcharge loads. The analysis has to be done for the worst combination of the following loads and moments.

4.8.1.1. Water Pressures

Water Pressures due to the differential water levels in the lock chamber and the saturation of the backfill would have to be considered wherever applicable.

4.8.1.2. Earth Pressures

The earth pressures due to the backfill including surcharge, if any, would be considered in the design of Walls.

4.8.1.3. Uplift

Uplift is considered as acting on 100% of the base area and is assumed to vary uniformly from the water level on one end to the other. Reduction in uplift pressure could be considered if sheet piles are provided at the upstream and downstream ends and also if grouting is done to reduce the seepage below the floor.

4.8.1.4. Gate Loads

The heavy concentrated loads from the miter gate, sector gate or any other type and their distribution should be considered in the designs.

4.8.1.5. Earthquake Force

The effect of earthquake due to both horizontal and vertical acceleration should be considered wherever applicable.

4.8.1.6. Tow Impact

Because of difficulties in maneouvering the vessels into the lock chamber, there could be impacts on the structures. These would also be taken into account in the analysis.

4.8.1.7. Line Loads

In locks, mooring facilities for both maneouvering and lockage purposes are provided. The line loads for both the purposes may affect the wall stability. Usually provision for line loads of 10 or 12 tons is sufficient. However, these are determined from model studies and the loads are taken as distributed load over a monolith of about 10 m length.

4.8.1.8. Wind Loads

Usually wind loads are omitted and wherever assumed, relevant wind velocity may be taken into account for calculating the load.

4.8.2. The lockwalls should be checked for stability against overturning, sliding and internal stresses. The required factors of safety should be obtained. In the design of gate monolith, stresses caused by torsional effects of the gates and other gate reactions to be resisted should also be calculated.

4.9. Figures 4.4 and 4.5 indicate typical layout plan and sections of a navigation lock.

CHAPTER 5

Head Regulator

5.0 Introduction

The head regulator or intake provided at the diversion structure has the following objects to fulfill : (*i*) Regulation of the supply of water into the canal or water conductor system for purposes of irrigation, hydel power generation, industrial or domestic water supply, etc., (*ii*) Prevention of high floods from entering into the canal and (*iii*) Control of the entry of silt into the canal. Hence it is very important that the head regulator is designed carefully for satisfactory hydraulic and structural performances since the success or otherwise of the project would depend on it also as it is the source at the head. The various aspects regarding the same are discussed in this Chapter.

5.1. Layout

From the point of view of sediment exclusion, the head regulator is generally aligned with its axis perpendicular to that of the diversion structure and located near the abutment. However, in some cases, it has been observed that keeping the axis of the head regulator oblique to the barrage/weir axis at an angle varying upto 110° or so gives better performance from sediment exclusion and smooth entry point of view. Hence aid of hydraulic model studies is generally taken for location and alignment of head regulator axis. The head regulator may be located on one or both the banks as per requirements.

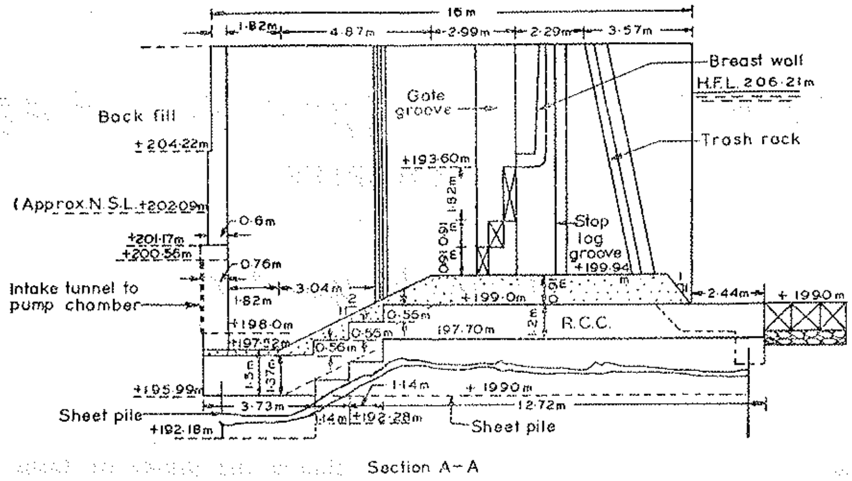
5.1.1. While the location of the head regulator adjacent to the abutment of the diversion structure is preferred, it may not sometimes be possible to locate it there due to topographical features such as hills, etc. In that case, the head regulator may have to be sited upstream near the periphery of the pond, but not very far from the main structure. If the discharge requirements are small, sometimes the head regulator or intake is provided in the form of an opening in the wing wall of the abutment, constructed at a suitable angle to the latter. A typical example of such a location is the intake of Iram Siphai Barrage which is shown in Figure 5.18. (Volume I)

5.1.2. The head regulator could be constructed independent of the abutment separated from it by suitable joints and seals or it can be monolithic with it, say in the case of gravity type of abutments of the main structure and the head regulator, with a valley line forming in between. The abutments of the head regulator themselves can be separated from its floor by longitudinal joints and seals or they can be made monolithic with the raft floor of the head regulator and the whole structure designed as a trough section. A typical example of the latter type can be seen in in Figure 5.13. (Volume I)

5.1.3. The regulation of the head regulator is provided usually by vertical lift gates. The gates can be a single or double or triple tiered one depending on the silt content of the river water and the purpose for which water is required to be diverted. This is further discussed in para 5.4.

5.1.4. The required discharge into the canal including about 10 to 15% of discharge for sediment extractors can be passed at pond level for which a gate controlled opening from the crest level to the pond level only is required to be provided. However, during high floods, water would spill over the gates and get into the canal if gates are provided only upto pond level. To reduce the uneconomical height of the gate right upto the high flood level due to higher cost of the gates, heavier machinery to operate them and high level of operating platform required, breast walls are provided instead above pond level and upto the high flood level plus a little free board or upto the top of the abutments.

5.1.5. Usually a road bridge is provided across the head regulator for vehicular traffic or for inspection and would be suitably connected to the bridge across the main structure. For the operation of the gates, a working platform across the head regulator would also be provided.



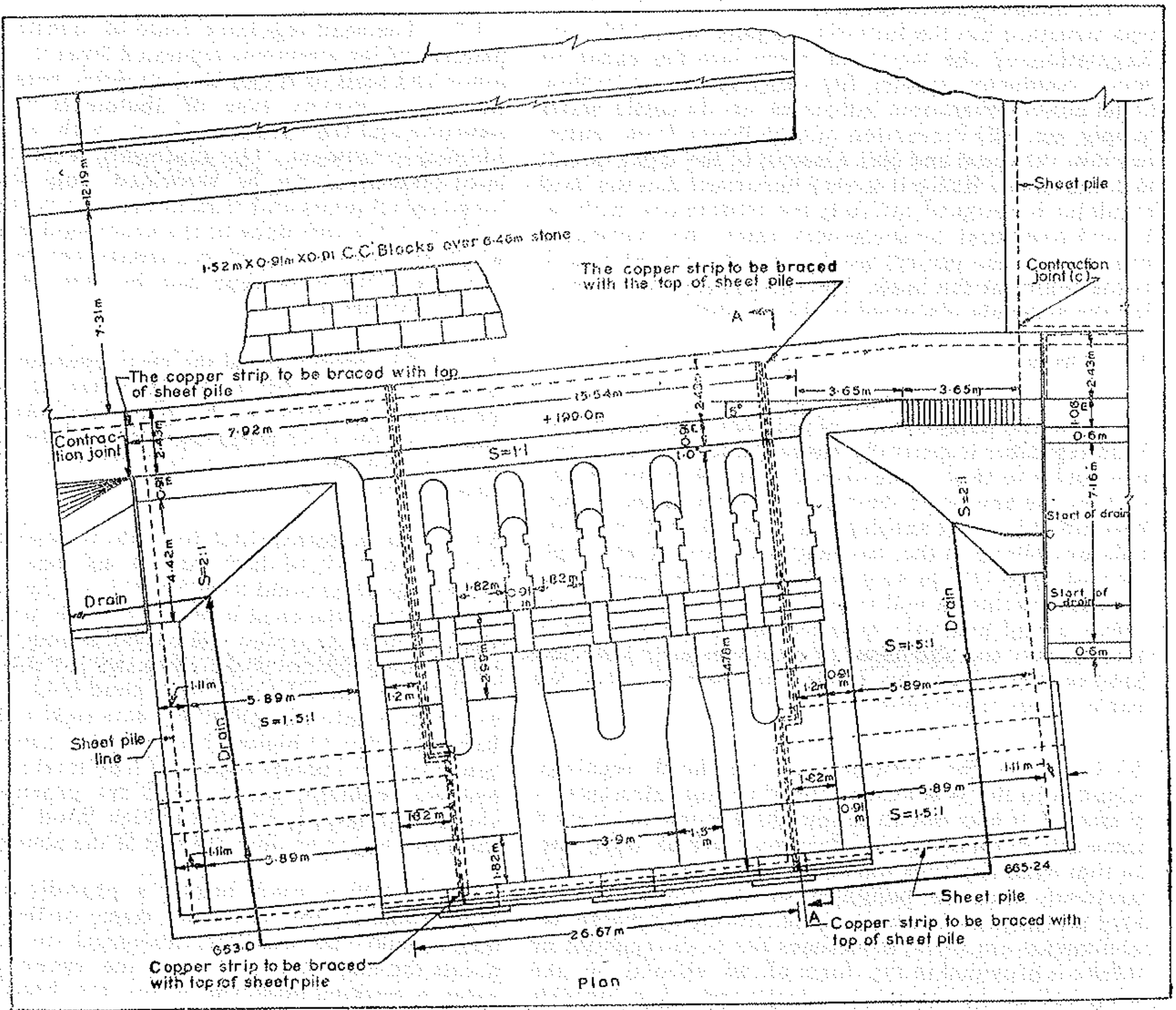


FIGURE 5.1 : Intake Details of Yamuna Barrage (Delhi)

5.1.6. In the case of head regulators feeding hydel channels, usually trash racks are provided in front of the gates and stoplogs to prevent floating debris from getting past the head regulators. This could be provided even in the case of irrigation canals if the floating debris are to be eliminated as much as possible.

5.1.7. Wherever intakes are provided for purposes of drawing water to be used in generation of thermal power, a sump is provided after the downstream glacis of the head regulator/intake. A typical example is that of the intake of Yamuna Barrage, Delhi. The layout of the same is shown in Figure 5.1. Similar arrangement could be adopted for domestic water supply purposes also.

5.1.8. Often times, the waterway of the head regulator may be more than the bed width of the canal downstream. In such cases, suitable transition would need to be provided beyond the downstream end of the head regulator to reach the normal section of the canal.

5.1.9. Sometimes, water may have to be diverted from the river to serve two main canals in different territories. It has to be analysed in such cases to determine whether a single head regulator with the combined discharging capacity and the two canals branching off after some common length could be had or two independent head regulators with two separate canals can be had. Apart from economical considerations, the operating conveniences including disputes likely to arise should also be considered. If two separate head regulators are to be provided, it would be desirable to determine their best locations and alignments with the help of model studies so that their hydraulic performances are not impaired.

5.2. Hydraulic Design

In the hydraulic design of the head regulator, the following items are decided : Crest level, waterway, profile of the floor and energy dissipation.

5.2.1. *Crest level* : The crest level and waterway of a head regulator are inter-related. For prevention of silt entry into the canal, the crest level should normally be kept about 1.2 to 1.5 metre above the crest level of undersluices if silt excluders are not provided and it would be about 1.75 to 2.5 m above if silt excluders are provided. The creast level coult be kept at the top level of the silt excluder tunnels or about 0.5 m higher depending on the silt charge.

5.2.2. *Waterway* : The waterway should be adequate to pass the required discharge through the head regulator without difficulty. In important structures, a standbye bay also could be provided to take care of any bay under repairs. The required waterway is calculated by using the formula.

$Q=C^d\ L_e\ H^{3/2}$ where C_d is the oc-efficient of discharge, L_e is the effective Width of waterway and H is the head over the crest. C_d is calculated from Drowning ratio values. For a head regulator, the value of C_d usually varies from 1.8 to 2 in British units for headless flow conditions. It is taken as about 1.7 in metric units. L_e is equal to $(L—K.\ n.\ H)$ where L is the clear waterway, K is a coefficient (usually 0.01 to 0.03) depending on the shape of the pier nose and n is the number of end contractions.

In the case of rectangular openings of intake, sometimes, submerged orifice formula could be used.

The spans of head regulators usually vary from 5 to 10 m. Hence as per the required waterway calculated, the number of spans and their widths can be decided. As already mentioned under layout, standbye bays could be provided wherever needed.

5.2.3. *Profile of the Floor* : While finalising the profile of the floor of the head regulator, the upstream and downstream cut off levels upstream floor level and length, downstream glacis, cistern level and length, and end sill level and length would be determined.

5.2.3.1. When the head regulator is located close to the undersluice bays of the main structure, the upstream cut off/sheet pile would be taken down to the same level as that of the undersluice bays. If it is located away from the main structure, it has to be taken down to the level governed by the usual considerations of scour on the upstream. The downstream cut off/sheet pile level would be governed by the considerations of exit gradient and scour explained for the main structure in Chapter 6.

5.2.3.2. The upstream floor is usually kept horizontal at the crest level, considerations for which have been given in para 5.2.1. The upstream floor would have to accommodate the grooves for trash rack bottom, stoplog and gates. It should also be fixed from the overall length required for a safe exit gradient which should be checked with canal closed and high flood on the upstream.

5.2.3.3. The downstream glacis is usually kept as 3(H) : 1(V) as for the main structure.

5.2.3.4. The cistern length and level are decided from the considerations for proper energy dissipation.

5.2.3.5. *Energy Dissipation* : Energy Dissipation is achieved through formation of hydraulic jump under different discharge conditions. For various gate openings with Pond level on the upstream, the discharge through the head regulator and the corresponding water level in the canal have to be worked out. From these values ; the cistern levels and lengths would have to be calculated and the governing values adopted for the profile. Additional energy dissipating devices such as chute blocks, friction blocks, end sill or dentated

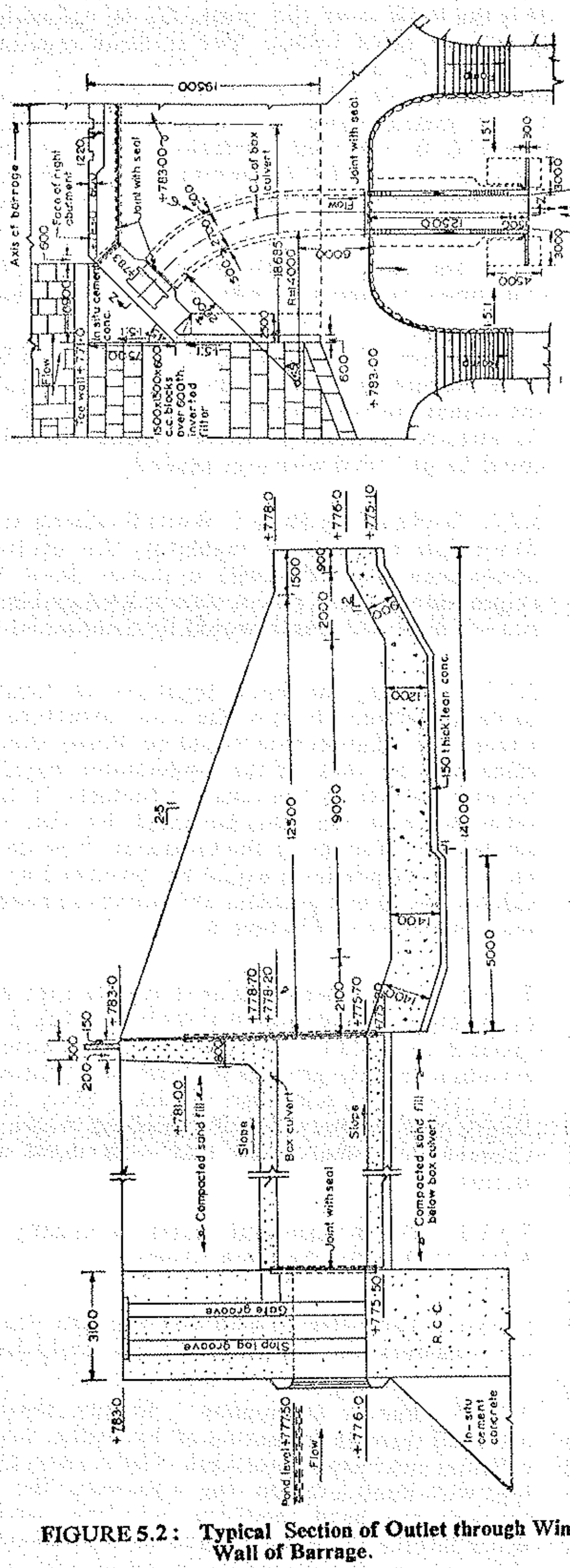


FIGURE 5.2 : Typical Section of Outlet through Wing Wall of Barrage.

end still, etc., could also be provided wherever necessary. For head regulators with small discharging capacities, additional energy dissipating devices except an end still may not be necessary.

5.3. Structural Designs

The structural designs of a head regulator are more or less similar to those of the main structure. As the canal is usually completely closed when the highest flood is passing in the river, it provides the worst static condition and the floor should be able to resist uplift pressures under this condition. These pressures are usually high and a gravity type of floor may not be economical. A reinforced concrete raft floor may prove to be economical in such places.

5.3.1. The breast wall of the head regulator generally consists of two parts viz., vertical stem and horizontal beam, The two parts can be constructed monolithic or separated by a joint and provided with a seal in between. The vertical stem would be designed as a slab spanning between the piers or pier and abutment, fixed at the two ends and loaded by the horizontal water thrust on the upstream side. The horizontal beam would be designed for bending in both the horizontal and vertical directions. The loads to be taken care of in the analysis include the self weight, horizontal water thrust on the upstream uplift acting below the beam and self weight of the vertical stem transferred at the beam eccentrically. Since the vertical stem is fixed to the piers and abutments, full weight of the stem may not be transferred to the beam. A fair proportion at the discretion of the designer may be taken as the transferred load. When the stem and beam are monolithic, the breast wall has to be checked for torsion also caused by the water trust. The area of the second stage concrete for the embedded parts should be neglected while analysing the beams.

5.4. Regulation of Gates

General precautions to be taken in the operation of the gates may be seen in Chapter 10. Wherever two tier or three tier gates are provided, for normal conditions when the silt charge is not much, the lower most tier of gates would be in operation. When the silt charge of water increases, the lower most tier would be kept in closed position and the other tiers of gates operated.

5.4.1. Wherever trash racks are provided to prevent the floating debris from entering through the head regulator, it is necessary that they are cleaned regularly so that the openings are not allowed to be chocked. Apart from increasing the differential head over the trash rack, the required discharge through the head regulator cannot be fed if they are allowed to get clogged.

5.4.2. Instructions regarding the closure of head regulator gates beyond a permissible silt charge of diverted water should be strictly followed.

5.5. Typical sections of an intake through the wing wall of a barrage are shown in Figure 5.2.

CHAPTER 6

Gates, Stoplogs and Trashrack

6.0. Introduction

Installation of suitable gates or falling shutters in diversion structures is an important item to be taken care of for the smooth and efficient operation of the project. Trouble free and smooth performance of these control equipment should also be ensured. The general types of gates extensively used now a days in barrages and head regulators are the fixed wheel gates and radial gates. For fish passes, fixed wheel gates are generally used and for navigation locks either fixed wheel gates or mitre gates are usually adopted. For proper maintenance of the gates, it would be necessary to have stoplogs which would enable the gates needing repairs to be isolated. For prevention of floating debris from flowing past the head regulator gates, trash racks would be installed in front of the gate and stoplog grooves. A general description of the gates, falling shutters, stoplogs and trash rack and also the hoist required to operate them and the design principles involved are discussed briefly in this Chapter.

6.1. Fixed Wheel Gates

Fixed wheel gates are the most modern type of vertical lift gate. In this type, the wheels are mounted on the side girder of the gate itself and move on the roller tracks fixed in the pier inside the gate groove. The main component parts of the vertical lift gate are (*i*) Structural steel skin plate (*ii*) Structural steel horizontal and vertical beams (*iii*) Wheels (*iv*) Seals and accessories (*v*) guide rollers or shoes (*vi*) Wheel tracks (*vii*) guides (*viii*) Seal bases and (*ix*) Anchor bolts for alignment.

6.1.1. *Structural Skin Plate* : The Skin plate is designed as spanning between the vertical stiffners or horizontal girders as the case may be. In cases where both vertical stiffners and horizontal girders are provided, the skin plate is designed in rectangular panel supported or fixed on all sides.

6.1.2. *Structural Steel Horizontal and Vertical Beams* : The horizontal girder spans between the vertical girders at either end of the gate and is designed as simply supported beam with the uniformly distributed load. The spacing of the horizontal beam is adjusted by trial and error that all the beams carry approximately equal water load. The end vertical girders are like continuous beams with concentrated loads transmitted by the respective horizontal girders. The girder is supported on a number of wheels. The bending moment in the girder and the shear are calculated by the usual methods.

6.1.3. *Wheels* : The wheels for the fixed wheel gates are usually made of cast steel or wrought steel and these require very careful design as they carry enormous loads. Each wheel assembly consists of the following component parts—wheel body, wheel pin or axle, wheel bearings and other accessories like springs, seal rings, etc., wherever necessary.

6.1.4. *Seals and Accessories* : In the modern designs, with the advance in technique of rubber manufacture, rubber seals are used in gates. They are made normally of music note type. The seal is always fixed so as to ensure a positive water pressure between the seal and the gate which enables the seal to bear highly on the seal seat and prevent leakage.

6.1.5. *Guide Rollers or Shoes* : Guide rollers or shoes are provided on either side of the gate to prevent the gate from drifting sideways when it is being lowered or raised, caused due to unequal pull. When there is no water pressure on the gate, there is also a possibility of the gate moving away from the wheel track. Guide rollers or shoes limit such movements within allowable tolerances.

6.1.6. *Wheel Tracks* : The wheel track is the load bearing member on which the wheels of fixed wheel gate transfer the water pressure. The purpose of the wheel track is to provide a smooth machined surface for the wheels to roll. Besides, the wheel track also transmits the heavy loads from the wheels in the concrete of the pier without allowing the stresses in concrete to exceed the permissible stresses in pier. The wheel track is divided into two parts namely, the path or wheel track and the track base.

6.1.7. *Guides* : The gaides are fixed inside the groove of the piers and these are normally built up of two angles kept back to back for guide shoes.

6.1.8. *Seal Bases* : The seal base comprises of a stainless steel seal seal screwed on to an angle which serves as a seal base.

6.1.9. *Anchor Bolts* : Since the fixed wheel gate moves in the gate groove within close tolerances, it is very essential to ensure that all embedded parts are aligned carefully within the specified tolerances. In order to obtain the tolerances, suitable means are to be provided in the embedded parts for alignment. Anchor bolts are usually provided to align the embedded parts in both the directions. It is a normal practice to leave blockouts or pockets behind the embedded parts with anchor bolts fixed at suitable locations in the first stage concrete. After the manufacture of the embedded parts, the same are fixed into the anchor bolts. Alignment is done by adjusting the nuts on the anchor bolts and after the alignment, the pockets are concreted.

6.1.10. Typical General installation of a fixed wheel gate is shown in Figure 6.1.

6.2. Radial Gastes

Use of radial gates in barrages is not very common. It has been provided in Kota Barrage across river Chambal. Some of the advantages of radial gates are (*a*) absence of grooves, in the piers which are unfavourable for the smooth hydraulic flow conditions and also debris, etc. (*b*) absence of wheels and wheel asseembly and (c) reduction in the hoist capacity compared to the fixed wheel gate. The following are the main component parts of the radial gate : (*i*) Skin plate—curved are of a circle (*ii*) Vertical stiffeners—curved are of circle (*iii*) horizontal girder (*iv*) end arms (*v*) trunnion castings (*vi*) bearing pedestal (*vii*) trunnion pin (*viii*) trunnion girder (*ix*) anchorages.

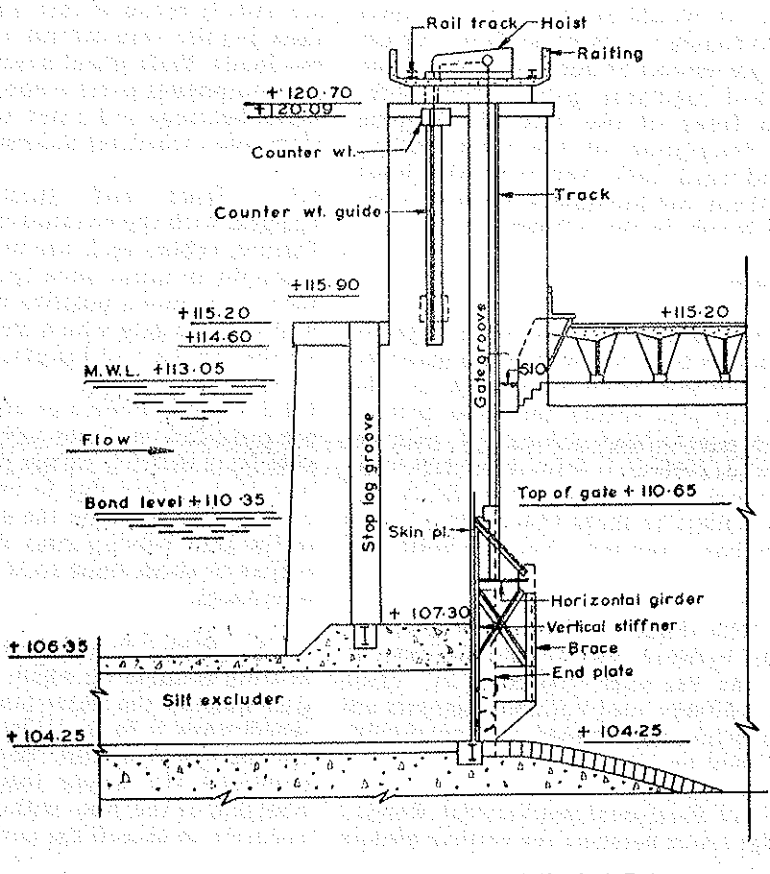


FIGURE 6.1 : Typical General Installation of Vertical Gate:.

6.2.1. *Skin Plate* : Curved Are of a Circle—The skin plate is designed as spanning between the vertical stiffeners.

6.2.2. *Vertical Stiffeners* : Curved Arc of Circle—The vertical stiffeners are spaced at convenient distances. Each vertical stiffener is designed as an over hanging beam spanning between the horizontal girder. The spacing of the horizontal girders is so adjusted that the maximum positive and negative bending moments on the vertical stiffener are equal.

6.2.3. *Horizontal Girder* : The horizontal girders are supported on the end arms at either end and transfer the load coming from the vertical stiffener to the end arms.

6.2.4. *End Arms* : The end arms serve as columns transferring the water load from the horizontal girder to the trunnion casting and hence into the piers. Each end arm is designed as a column with L/R ratio equal to or less than 180.

6.2.5. *Trunnion Casting* : The trunnion casting is a circular member made of cast steel against which the end arms converge and transfer the load centrally to the pin.

6.2.6. *Trunnion Pin* : The trunnion pin in the radial gate is a vital part and it carries enormous load. The pin is designed as a simply supported member over the hub and carrying a uniformly distributed load from the trunnion hub and bearing.

6.2.7. *Bearing Pedestal* : The bearing is made of cast steel in which the pin is suitably housed, While erecting the gate, it is extremely important that the pin is aligned properly. In order to enable this alignment, a suitable arrangement is made in the bearing pedestal.

6.2.8. *Trunning Girder* : The trunnion girder transfers the load from the pin of the radial gate into the concrete of the pier.

6,2.9. *Anchorages* : Anchorages transfer the load from the trunnion girders into the piers. For small radial gates used in some regulators, the elaborate anchor girder and anchorages can be eliminated by supporting the bearing pedestal on a concrete projection from the pier.

6.3. Mitre Gates

Mitre gates are one of the types of gates usually adopted for locks. They are made in pairs, pivoted on a vertical axis in a recess in each lock walls. The combined width of the pair of gates is greater than the width of the lock and the outer vertical edges of the gates meet in an angle pointing upstream. The thrust of water against the gates is transferred along the horizontal beams of the gates to the walls.

6.4. Falling Shutters

Among the different types of falling shutters Fouracres' falling shutters and Ashford's patent drop shutter are generally used over weirs.

6.4.1. In the Fouracres' Falling shutter, each shutter, is maintained in the upright position by tie bars, one end of each bar is attached by a movable pin joint to an eye bolt anchored in the weir crest upstream of the shutter and the other end is similarly attached to a point on the shutter a little below the centre of pressure of water when the shutter is on the point of overturning. When water rises to such a level that the moment of water pressure about the points of attachment of the tie bars becomes sufficient to overturn the shutter it falls into the position and lies flat on the crest, offering no obstruction to the flow. The shutters will then have to be raised by hand when the depth of water passing over them is not more than 0 5 metres. In this type of falling shutters, any obstruction or accumulation of silt on the upstream side of the shutter is likely to prevent it from acting and it is unsafe to staunch leakage under or between shutters of this type by depositing silt upstream. Secondly, individual shutters are very likely to be overturned by wind and wave action at times when the general level of water is considerably below the level at which it is desired that the shutters should fall. Typical section of this type of shutters is given in Figure 6.2.

6.4.2. Ashford's Patent Drop Shutter is another type attached to the weir crest by hinges along its lower edge. Each shutter is held up by a tie rod or by a strut which can be released by the operating staff. Each shutter is supported by a single strut, one end of which is connected by a pin joint to a plate fixed in the weir masonry and the other end slides in a groove in the back of the shutter and cannot leave the groove. When the shutter is raised, the end of the strut is retained in the corresponding position by the short end of the lever bar. The lever bar is locked in the horizontal position retaining the strut by making a jointed end on the longer limb of the lever and slipping this end into a notch in a shutter frame, the hinged end of the lever projects beyond the frame so that the next shutter when falling may push it out of the notch and thus unlock the lever and release the end of the strut. The shutters are installed in one or more seats working from one end and all the shutters of each set are released in succession by the falling of the next adjacent shutter, only the end shutter being released by hand. Typical section of this type of shutter is shown in Figure 6.2.

6.4.3. The falling shutters which lie down horizontal with the passage of floods would be raised by shutters set after set by means of a travelling machine called "plough". The plough moving forward on its track on the weir catches up the roller in the middle of the free end of the shutters. This roller moves along over an inclined track in the plough so that as the plough goes forward, the shutter rises to its vertical position. The strut arms in rear are then pressed down to their final position which is nearly 6 mm. below the dead centre line. Thus shutter after shutter is lifted up. For tripping any particular set, oil from the valve houses is pumped on to any particular master shutter plunger mechanism which lifts the strut arms out of its normal position below the dead centre line and the water pressure on the face of the shutter trips it down. In falling, this drops down the other shutters of the set one by one due to the connecting shaft and clutch arrangement.

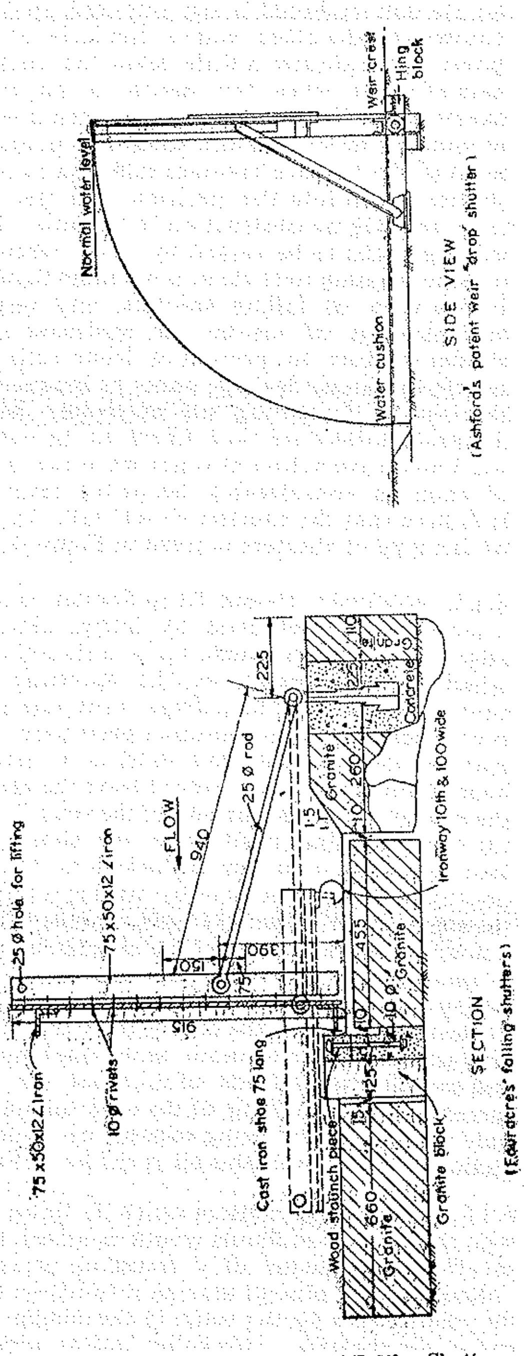


FIGURE 6.2 : Typical Sections of Falling Shutters.

6.5. Stoplogs

For repair of the gates of the barrage or head regulator, it would be necessary to stop the flow through the bay in which the gate is to be repaired. For this purpose, stoplogs are used. These are horizontal cross pieces placed across the waterway and slide in vertical grooves of the piers and abutments. Previously stoplogs used to be made of wood, but now-a-days flitched beams with H – Sections of mild steel filled in by teak packings bolted to the horizontal web are being used. Usually the same gantry crane or hoist used for lifting or lowering the gates is employed for operating the stoplogs also. Depending on the number of bays of the barrage and regulator, one or more sets of stoplogs are stored either near the abutments or between piers in the midbays over brackets anchored to the piers,

6.6. Trash Rack

Trash racks are provided in front of the stoplog grooves of the head regulator to prevent floating debris from entering into the canal through the head regulator. This is a framed structure of steel with horizontal and vertical beams and struts. They would be designed as beams or columns as the case may be to resists the differential water pressure caused due to the choking of the waterway of the trash rack. The differential head may vary from 1 to 3 m depending on the amount of floating debris and the effective cleaning time of the trash rack. The trash rack is provided at an inclination of about 10-20° to the vertical and installed in the grooves of the piers and abutments provided for the purpose. Sometimes it may be necessary to prevent the entry of floating grassy weeds into the head regulator as in the case of an intake feeding cooling water to a thermal power station. In those cases, in addition to the trash racks with bigger openings, finer meshes may also be used in the stoplog grooves.

6.7. Hoists

Hoists are provided for operation of gates to regulate the flow through the barrage or regulators. If there are a number of gates to be operated, either travelling or gantry cranes may be used for their operation. However, the period required for the operation of the gates is greater. Hence, often, individual hoists are provided. Different types of hoists are used for operating gates depending upon the hoist capacity required and the operating conditions. The following are some of these types – (*i*) Screw lift hoist (*ii*) Rope-drum or Chain-sprocket hoist (*iii*) Hydraulic hoist (*iv*) Automatic hoist with float operation.

For details of the hoists, relevant manuals may be referred to.

CHAPTER 7

Model Tests

7.0. Introduction

In the previous chapters, theoretical considerations for the selection of site, type of structure and design aspects of the diversion structure and its appurtenant works have been discussed. There are a number of of complex factors which cannot be properly visualised by a designer. Hence the aid of model tests is, therefore, invariably taken to evolve the optimnm design which is hydraulically and structurally sound and economical. There are different types of models such as 2-Dimensional, 3-Dimensional, etc. The different types of model techniques are not discussed in this Chapter. The utility of model tests in so far as diversion structures are concerned, the data required for the same and their limitations are only outlined in this Chapter and a few examples of model tests are also presented.

7.1. Utility of Model Tests

In so far as the diversion structures and their appurtenant works are concerned, model tests are utilised for study of one or more of the following aspects :

(*i*) Selection of site and orientation

(*ii*) Type of diversion work

(*iii*) Coefficient of discharge, afflux, pond level length of waterway and crest level

(*iv*) Energy dissipation devices

(*v*) Location of abutments

(*vi*) Distribution of discharge in different channels of the river

(*vii*) Backwater studies

(*viii*) Effect of structures located nearby including special problems

(*ix*) General layout and flow pattern

(*x*) Concentration of flow and scour pattern

(*xi*) Gate Regulation.

(*xii*) Design of components of structure such as undersluice, divide walls, head regulator, silt excluding devices, gates, etc.

(*xiii*) River Training Works.

(*xiv*) Location, waterway and designs of Fishpass.

(*xv*) Location, alignment and design of Navigation lock

7.1.1. For any structure, it may not be necessary to test all the above mentioned aspects. A few things can be decided upon by the designer himself based on experience and general study of the behaviour of similar structures.

7.2. Data required for Model Tests

For laying the model before it is 'run' and tested, certain data are to be furnished to the testing officer. A list of data generally required for the model tests is presented as Appendix 7.1.

7.2.1. A correct model analysis of the problem depends upon the correct laying of the model which again depends upon the correctness and quantum of data furnished. When the model is being laid in a research station, it is always desirable that a field officer, intimately associated with the site conditions, is present there so that the transfer of data to the model is accurately done. When the laid out model is 'run', he can observe the flow pattern and wherever needed, the model can be corrected so that a true depiction by the model is obtained.

7.3. Reference to Model Tests

It is necessary for the designer to intimate the model testing officer about the various aspects to be studied on the model. In other words, he has to give the "terms of reference" for the model tests. This would contain one or more of the variovs aspect mentioned in para 7.1. Copies ofthe hydraulic design calculations and preliminary drawing prepared on that basis would need to be furnished to the testing officer.

7.4. Model Test Report and follow up Action

After the necessary model tests are over, the observations, results, conclusions and recommendations would be presented in specific notes along with relevant photographs if any. These would be studied and the recommendations, wherever acceptable, would be incorporated in the final designs and drawings.

7.5. Illustrations of Model Test Results

A few examples highlighting the utility of the model tests leading to suitable modification of the original theoretical design are given below :

7.5.1. *Project Features of Kosi Barrage*

7.5.1.1. CISTERN DESIGN

For the spillway portion, the calculated theoretical cistern level for the worst condition of flow was El. 221.00 (ft). However, from model tests, it was found that a cistern at El. 223.5 (ft), (2.5 ft) higher than the theoretical one) and 89.5 ft long and provided with 2 rows of friction blocks each of 5 ft height and end still of 2.5 ft height provided a satisfactory energy dissipation. Similar improvement was recommended for the undersluice portion also.

7.5.1.2. DIVIDE WALLS

The lengths of the divide wall forming the right and left pockets, according the original design, were 240 and 650 feet respectively. The optimum lengths, as suggested from the model tests, from the point of achieving better sand exclusion, were found to be 288 ft (increase or 48 ft) and 400 ft (decrease of 250 ft).

7.5.1.3. SAND EXCLUDERS

The two bays of the right pockets provided with excluder tunnels were observed from the model tests to be adequate. However, in the case of left pockets four number of excluder bays were found essential instead of two bays, from the point of efficient flushing out of bed sand and thus prevent it otherwise from getting lifted over the top of the excluders and being drawn into the East Kosi Power Canal.

7.5.1.4. HEAD REGULATORS

Two alternative designs were suggested in the original design. The first was with off-take normal to the barrage axis and the second was with an inclined off take making an angle of 102° 30′ with the barrage axis. The latter was found to be satisfactory in the model from the consideration of sand exclusion.

7.5.1.5. GUIDE BUNDS

The alignments adopted at site for the upstream guide bunds were different from those recommended earlier from model tests. Corrective measures viz., (*i*) increasing the radius of curved heads of right guide bund from 500 to 1700 ft, and taking it back through 90° and (*ii*) increasing the angle of sweep for the left guide bund curved head from 90° to 120° were seen to function satisfactorily in the model

7.5.1.6. CENTRAL UNDERSLUICES

Model tests indicated that original proposal of providing the central undersluices were not satisfactory since its effect was found to be only local and not very marked and as such the proposal was not recommended.

7.5.2. *Location of proposed Tonle Sap Barrage (Kampuchea)*

In the original design, the barrage was located in the western channel. In the model, it was found that with this location, there would be increase in the maximum velocity at Kompong Chhnang Port. The barrage was, therefore, ultimately proposed to be located in the eastern channel when it was found in the model that the flow condition at the port would not be disturbed.

7.5.3. *Project Features for Bidyadharpur Barrage on Salandi River*

7.5.3.1. The end sill of the barrage was recommended to be lowered form El. 111 (ft) to El. 107 (ft) (lowering by 4 for ft) proper energy dissipation.

7.5.3.2. The divide wall was recommended to be omitted as it was found to serve no purpose.

7.5.3.3. The length of the guide bund was reduced in the portion where velocities were low. This portion of the guide bund was recommended to be converted in to afflux bund.

7.5.4. *Project Features for Sone Barrage*

7.5.4.1. The original design of Sone Barrage consisted of 57 weir bays with 8 undersluices on the left and 4 on the right. In the recommended design. the spans in the weir portion were kept constant but the number of undersluices were made six on each side.

7.5.4.2. The original design provided for a cistern length of 92 ft. It was recommended to be reduced to 80 ft (reduction of 12 ft).

7.5.4.3. The lengths of concrete block protection and loose stone apron were recommended to be reduced from 89 ft and 90 ft to 47′6″ (reduction of 41′6″) and 60 ft (reduction of 30 ft) respectively.

7.5.5. *Layout of Trisuli Diversion Works*: Original design consisted of a gated structure 468 ft long between abutments having 11 spans of 30 ft each of spillway and 3 spans of 20 ft each of undersluices. Model tests suggested the structure instead should consist of 3 spans of 20 ft each of undersluices, 3 spans of 20 ft each of river sluices and a weir without shutters in the remaining length.

7.5.6. *Gandak Navigation Lock*: Model tests were conducted to determine the location and alignments of upstream and downstream navigation locks, their dimensions, approaches, hawser forces, filling and emptying arrangements, arrangement (layout) of navigation channel crossing the Gandak Right Main Canal, realignment of downstream guide bund of the barrage, etc.

7.6. Limitation of Model Tests

Model tests are only aides in the finalisation of structures. They have their own limitations like difficulties in the reproduction of silt content of water, etc. Hence for some of the tests, the results could give only a qualitative idea and quantitative idea may be only approximate. Hence the designer needs to incorporate the recommendations of the model tests carefully.

APPENDIX 7.1.

Field Data Required for models of Barrages and/or their Appurtenant Works

Sl. No.	*Item No.*	*Type of Data*	*List of Data*
(1)	(2)	(3)	(4)
1	1	Report	This should include. (1) Enunciation of the problem, its history & probable causes with additions & modifications of works, if any. (2) Account of previous remedical measures taken if any, their details and behaviour. (3) Report of any project on the river in or near the problem reach which affects the river regime such as dam, weir, bridge, causeway, embankments, etc. (4) Hydraulic design calculations of existing & proposed structures, if any, in and near the problem reach of river. (5) Photos depicting behaviour of the river during floods.
2	2	Survey Data	(1) Index plan (normally to a scale of $1:2.5\times10^5$ to $1:10^6$ depending on size) should show the reach under consideration, mentioning State, Distrit, important towns in the vicinity to help location, tributaries joining the parent river, catchment area, etc. (2) Survey Plan (normally to a scale of 1:10000 to 1:50000) for the reach to be reproduced in the model (length of the reach to be reproduced may be from 2 meander lengths upstream or to end of backwater whichever is more. On the downstream side, model would extend to at least 1 meander length). **Note**: Similar data of alternative sites if proposed. This plan should show: (*a*) a closed traverse covering entire reach to be modelled. (*b*) latitudes and longitudes, (*c*) cold weather channel, (*d*) formation of rapids, pools, etc.

Appendix 7.1 (*Contd.*)

(1)	(2)	(3)	(4)
			(*e*) position of existing and proposed structures like bridges, dams, weirs, barrages, ghats, spurs, revetments, etc.,
			(*f*) location of the problem area,
			(*g*) the shoals and submersible land should indicate the nature of vegetation,
			(*h*) the various channels in case of braided river.
			(2A) An aerial survey plan, if available.
			(3) Cross-sections at suitable intervals (This will depend on model scales and will be specified in individual cases). These should extend beyond the limits of spills and be tied to the traverse survey. On the cross-sections, HFLs with date of occurrence should be marked. Positions of sections with their zero chainages should be indicated on the survey plan referred in (2) above.
			(4) Plan showing contours or spot levels over the spill area. Contour interval may be varying from 0.5 m in case of flat terrain to 1.5 m for steep country. Latitudes and longitudes should also be marked on this plan.
			(5) Plan of previous river surveys indicating the changes in the river course, if any.
			(6) Details of existing and proposed structures like bridges, dams, weirs, barrages, spurs, revetments, etc. All levels should be connected to G.T.S. bench-mark.
3	3	Hydraulic Data	(1) Daily gauge & discharge data at all existing sites for one or more flood seasons. If stream gauging sites are absent, a minimum of three new stations should be located, one each at either end of the reach to be modelled and one in the middle and data collected and furnished for at least one flood season. The positions of all gauge stations should be shown on plan in (2) above and their co-ordinates given.
			(2) Bata of existing discharge site for preparation of gauge discharge curve. If no discharge site exists, it should be established based on the following considerations :
			(*a*) Discharge site should not be too far away from the reach under study.
			(*b*) There should not be much intervening catchment and spills between the discharge site & upstream end of the model.
			(*c*) Site may be beyond the backwater region caused by structures, tributaries, etc.
			(*d*) A new discharge site should preferably be located at the Central Gauge Station within the model reach.
			(*e*) River cross-section at discharge site, indicating nature of river bed (Sandy, boulders or rocky) and sample discharge calculations should be sent. This cross-section must coincide with one of the dry weather cross-sections.
			(3) Annual H.F.Ls. with dates of occurrence.

Appendix 7.1 (*Contd.*)

(1)	(2)	(3)	(4)
			(4) Maximum flood on record with the date of occurrence.
			(5) In case the reach of the river to be modelled is tidal, information regarding H.T.L. and L.T.L. of spring as well as neap tides, both during freshets & dry season.
			(6) R.Ls. of flood marks on structures & depth of spill flow & its course.
			(7) Discharge distribution in various channels at important stages.
			(8) Catchment characteristics in regard to topography, rainfall and run-off. All gauge and discharge sites should coincide with cross-sections and should be marked on plan referred to in para (2) under Survey Data.
			(9) Gauge discharge curve of the river downstream of the structure under retrogressed and unretrogressed water-levels.
			(10) Proposed pond-level or levels.
4	4	Sediment Data	(A) Bed Material
			(1) A minimum of three representative bed samples, 1.5 kg each, across a section one at either end and one in the deep channel, avoiding rock out-crops. Samples from three such sections, one at either end and one in the centre of the river reach under consideration.
			(2) Bore-hole data along the alignment and on a normal section down to 150 m for determining type of energy dissipation to be provided.
			(3) Bed material from the banks and the bed of the river both from the flowing and dry channels may be collected along the cross-section according to the variation in the deposits of the bed material and "not necessarily from 3 points".
			(4) These samples may be analysed for determination of the size distribution and mean diameter.
			(B) Bank Material
			(1) Material from which both the banks are made is to be collected on the same sections from which bed samples have been obtained.
			(2) Information regarding nature of banks-whether of uniform material, stratified, rock outcrops, kankar, etc., should be marked.
			(C) Suspended Sediment Data
			(1) If an integrating sampler is not available, suspended silt sample should be collected using a bottle sampler near the central Gauge station in the reach. Samples should be collected at medium and high flood stages at 0.5 or 0.6 depth subject to verification by actual observations over a vertical in the deep channel.
			(2) Construction and size distribution of suspended silt samples on a section through head regulator. Sampling should preferably be made at a minimum of five flood stages, on three verticals and at 30 cm below the surface, 0.5 or 0.6 depth (subject to verification by actual observations) and 30 cm above the bed on each

Appndixe 7.1 (*Contd.*)

(1)	(2)	(3)	(4)
			vertical. These sampling points should be located one at either end at 1/6 width of the river from the banks and one at the centre.
			Position of the sampling verticals should be marked on the plan.
			Note : Position of bed and bank material samples, bore-holes and the sampling vertical should be marked on the Plan.
5	5	**Other Data**	*(1) Lines of flow at specific flood stages low, medium and high.
			(2) General layout plan of the barrage or weir and its appurtenant works.
			(3) Regulation procedure at different flood stages including gate opening in the various spans, details of scouring operations, etc.
			(4) Plan, L-Section & Cross-section showing all details of the works.
			(5) L-Section and representative cross-section of the canal for the first 1.5 km.
			(6) Detailed drawings and design calculations of appurtenant works like : (*i*) Guide Bunds (*ii*) Divide Walls (*iii*) Sand Exclusion Devices (*iv*) Pockets (*v*) Head Regulator (*vi*) Canal (*vii*) Undersluices (*viii*) Piers and Gates
6	6	Specific Terms of Reference	Exact terms of reference should be stated by the sponsoring authority.
			Note : In case the river reach to be modelled includes tributaries or branches, similar data for the concerned reaches.

***DATA** marked thus may be omitted in the case of unimportant Problems.

CHAPTER 8

Instrumentation

8.0. Introduction

The diversion structures like barrage and weir are generally designed on the principles governing the percolation of water below the foundation of the structure. The floor of the structure is suitably designed either as a raft or gravity section to be safe against the uplift pressures created. Reinforcement is also provided sufficiently such that the permissible stress limits are not exceeded. Due to the various assumptions to provide for the 'unknowns' in the design, sometimes the designs become a little conservative. Hence in order to know the 'health' of the structure under different loading conditions and also to know the progressive behaviour of the structure, we need to have instrumentation in the structure. Most of the hydraulic structures are indeterminate in nature due to the three dimensional aspects. When they are analysed two dimensionally in the designs as simple beams and columns, the results sometimes lead to uneconomical provisions to be made. By actual observations of their behaviour through instrumentation, the designs can be improved and economy effected. By having a continuous record of the observations with the instruments installed, we can locate the distress spots in the structure and take remedial measures to make the structure safe. Apart from this, the observations help reduce the 'unknowns' and place the future designs on sounder footings.

8.1. Instruments Required

In barrages and weirs, generally three types of instruments are installed. For the measurement of uplift pressures below the structure, pipes are installed. To measure the pressures to which the foundation is stressed, Soil stressmeters are installed. For measuring the stress in the reinforcing bars due to various loading conditions, Reinforcement meters are installed. Other instruments which are also installed are settlement gauges and gauges for measurement of lateral earth pressures acting on the abutments.

8.2. Planning of Instrumentation

In order to achieve the objective of instrumentations as mentioned in Chapter 1 on Introduction, it is of extreme importance that a systematic and complete plan of various observations/measurements must be prepared for obtaining as complete an information as is possible regarding the various loadings influencing the structural behaviour of the structure and factors indispensable to interpretation of the results of such measurements. Measurements should be planned for the most important zones of the structure. Planning of measurement should be made taking into consideration the results of the analytical and experimental (model) investigations, as also of the studies of the foundation and its specific problems. Various types of measurements made on previously built similar structures can aid in planning the necessary types of measurements for obtaining the needed information. It is essential that the various measurements made on the structure should be so planned as to provide information, not only of the individual components of structural action, but also of the integrated structural performance of the structure and its foundation.

8.2.1. The nature of measurements to be carried out on any structure will depend upon the size and importance of the structure. However, while planning the instrumentation, the cost of the equipment, of the observing personnel, and of the processing of observed data and interpretation of the results should be considered in relation to the cost of the structure and its situation. Limitation of cost of instrumentation in a structure may be a determining factor in deciding the types of measurements to be made on a given structure.

8.2.2. Once it has been decided to instal instrumentation, it should be ensured that the methods of measurements, instruments, staff employed for observation and processing of observed data and interpretation of results must be of a calibre needed to achieve useful results under the inevitable variability of conditions.

8.3. Unlift Pressure Pipes

Of the instruments generally installed in barrages and weirs, uplift pressure pipes are the most common. These are used along with a bell sounder for measurement of uplift pressures developed at the point of tapping (1) Figure 8.1.

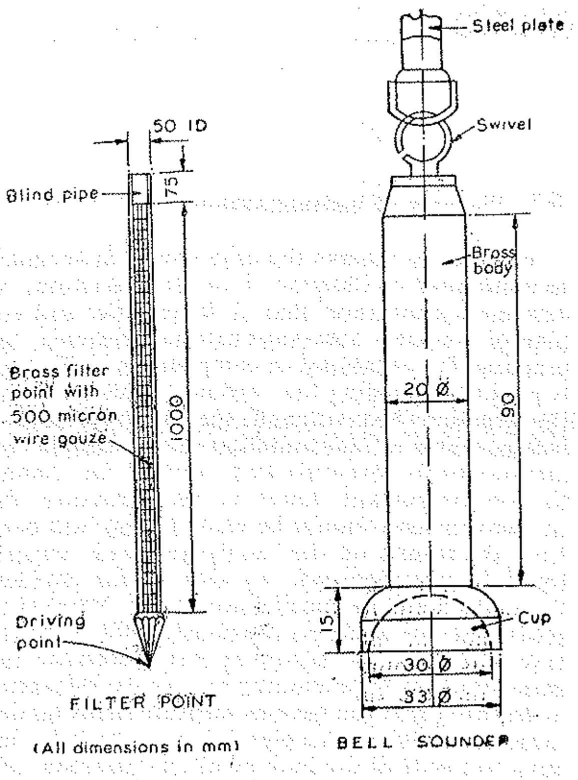


FIGURE 8.1 : Filter Point and Bell Sounder.

8.3.1. **Description** : The main instrument placed at suitably selected points consists of brass filter points of 50 mm inner 40 mm. G.I. pipes to suitable stand pipes located on the superstructure for measurement of water level. The filter point is fitted with a driving point at one end and a threaded blind pipe of 50 mm diameter and 75 mm length at the other end, 100 cm long, 500 micron wire gauge strainer is fitted in between and around the filter point to serve as the filter.

The water levels in all the stand pipes are read by a bell sounder lowered into the pipe by a steel tape or by electrical devices. The bell sounder consists of a solid brass rod about 90 mm long and 20 mm. in diameters ending in an inverted cup of 30 mm diameter. The steel tape is attached to the swivel screwed on to the upper end. The moment the cup of the sounder hits the water surface within the pipe, a definite 'plop' sound can be heard.

8.3.2. **Installation** : The filter points are generally laid horizontal where excavation permits. Otherwise they are driven to proper level. Where the depth is too great or where the soil is hard so as to damage the filter point while driving, the filter point and connecting blind pipe may be inserted in a bore hole of 100 mm diameter. Depending on the condition of the sub-soil material, graded filter material around the filter points may or may not be provided. Alternatively porous tube piezometers may also be provided. Certain precautions like leak-proof connections, absolute verticality of pipes, spacing, anti-corrosion, clearing of choking in the pipe, etc., have to be observed. For more details on these relevant Indian Standard Codes may be referred.

8.3.3. **Location** : The pressure tapping points are divided into three groups, namely, (*i*) along and immediately beneath the horizontal or sloping floors, (*ii*) at different points along the deep vertical cut-offs and (*iii*) at different depths in the sub-soil. Under (*i*) the tapping points are located under the floor at the upstream and downstream ends, immediately upstream and downstream of each vertical cut-off and at other intermediate points at regular intervals. These points enable us to know the distribution of uplift pressures beneath the floor. The tapping points along the faces of the deep. Vertical cut-offs enable the determination of the effect of the depth and spacing of vertical cut-offs and of the stratification of the sub-soil on uplift pressures. The tapping points at the bottom of the vertical cut-off at the downstream end of the structure enables computation of exit gradient. Depending on the importance of the structure, tapping points along the abutments and at a number of intermediate sections between the abutments at suitable intervals may be located.

8.3.3.1. For typical locations of the unlift pressure measuring pipes, (1) Figure 8.2.

8.3.4. **Observations** : The frequency of observations depends on the local requirements. However, for watching the stability of the structure, once a week for the key points and once a fortnight for other points would be adequate. When the daily water level changes are more than 10 m, Observation may be made daily.

8.3.4.1. Apart from the main observations of water level on the stand pipes discussed in a subseqnent para, observations will have to be made regarding upstream and downstream water levels, shade temperature, temperature of river water at suitable depth where it is approximately constant, temperature of sub-soil water and the depths of sediment on the upstream and downstream floors and if possible, the soil characteristics of the sediment.

8.3.4.2. The water levels in the stand pipes are measured by the bell sounder. The depth of water at the moment of hearing the 'Plop' of the sounder on

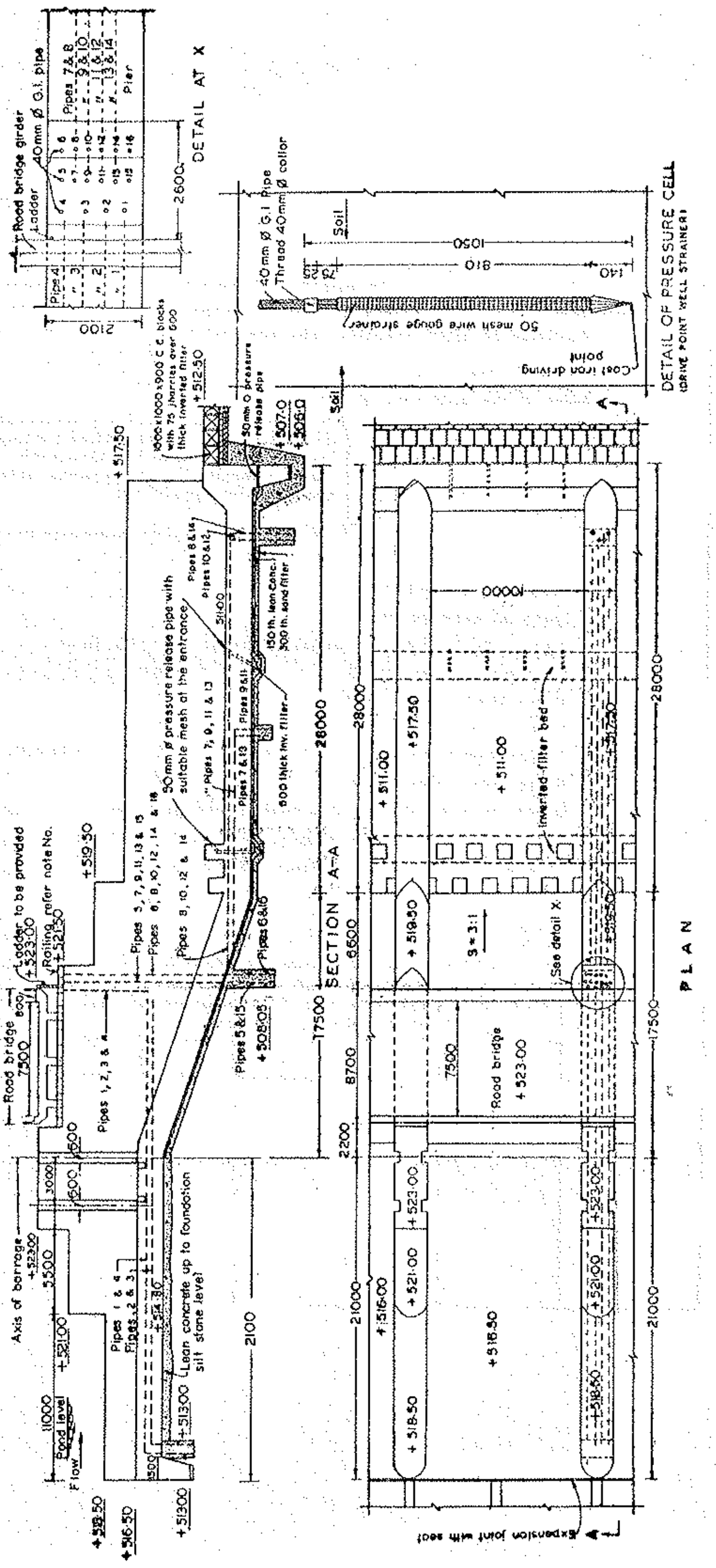


FIGURE 8.2 : Typical Location of Uplift Pressure Measuring Pipes.

FORM I

Register of uplift pressure pipe observation

Name of the river :

Name of the Hydroulic structure :

Give sketch of cross section along the line of pipes of the structure showing details indicated in the note at end of this proforma

Name of observer :

Date of observation :

Approximate time of observation .... Hr. to .... Hr.

Upstream water level : (m)

Downstream water level: (m)

Head H = (m)

Shade temprature, Maximum °C Minimum °C

River water temprature : °C

Depth of sediment on upstream pervious floor : (m)

Depth of sediment on downstream pervious floor : (m)

Line No. Total width of pucca floor : (m)

Pipe No.	Distance from upstream end of pucca floor	Reduced level of bottom of pipe	Reduced level of bend of pipes, if any
1	2	3	4

FROM II

Pipe No.	Reduced level of top of pipe m	Pipe water temprature C°	Depth to water in pipe m	Reduced level of water in pipe m	P m	O= (P/H)x100		Remarks
						Designed	Observed	
1	2	3	4	5	6	7	8	9

Note: Following features may be shown on the sketch

(a) Foundation profile

(b) Uplift pipes with their number (Reduced levels of bottom and tops of pipes bends if any, distance from upstream end of pucca floor and

) Stratification of the substrata

FIGURE 8.3. : Typical Proforma of Record.

touching the water surface inside the pipe is measured accurately and the reduced levels are obtained. Due to slow rate of movement of sub-soil water, the water in the stand pipes takes some time to deplete or recuperate corresponding to a fall or rise in the pressures. Hence the time log has to be determined and taken care of while observing readings.

8.3.5. **Recording** : Observations have to be recorded on the following :

(*a*) Date of observation

(*b*) Upstream and Downstream water levels

(*c*) Total Head H = upstream WL − downstream WL.

(*d*) Maximum and Minimum shade temperature

(*e*) Temperature of river water

(*f*) Temperature of sub-soil water (in selected pipes)

(*g*) Depths of sediment on upstream and downstream floors

(*h*) Water levels in all the stand pipes

(*i*) Residual pressure in each pipe P = Difference between WL in the stand pipe and downstream river WL

(*j*) Velocity potential percentage $\phi = \frac{P}{H} \times 100$

8.3.5.1. A typical proforma of record is given in Figure 8.3.

8.4. Soil Stress Meter

The soil stress meters are provided at the foundation level to measure the effects of external loads applied on the structure. These are similar to the stress meters in concrete. There are two categories of instruments used for the purpose, namely contact pressure cells and total pressure cells. In one of the types of contact pressure cells, called the twin plate type, the earth pressure is taken by a stiff plate to transfer the earth reaction to a beam which deflects under the load and is in turn supported by another plate. The twin plate unit bears on the surface of the rigid structure. The deflection of the steel beam is measured by vibrating wire instruments.

8.5. Strain Meter

The stress in the reinforcement steel is calculated with the use of strain meters. After finding out the strain, the value of stress in the bar can be calculated knowing the elasticity modulus of steel. There are some wire stress gauges like CV−28-2 which also can be used to measure stress directly on steel.

8.6. Figures 8.4. and 8.6. give typical details of location of various instruments and their recording station in a barrage.

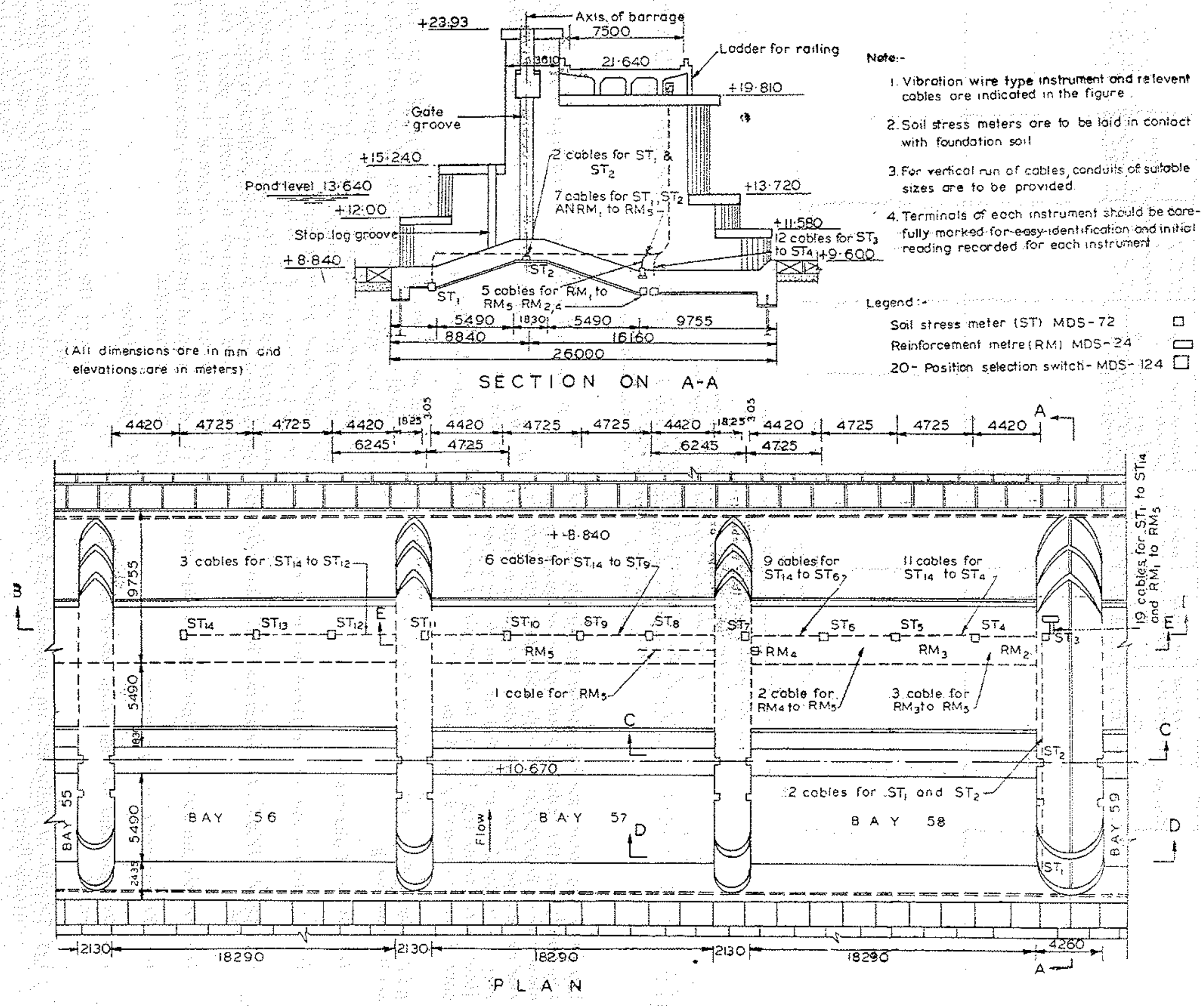
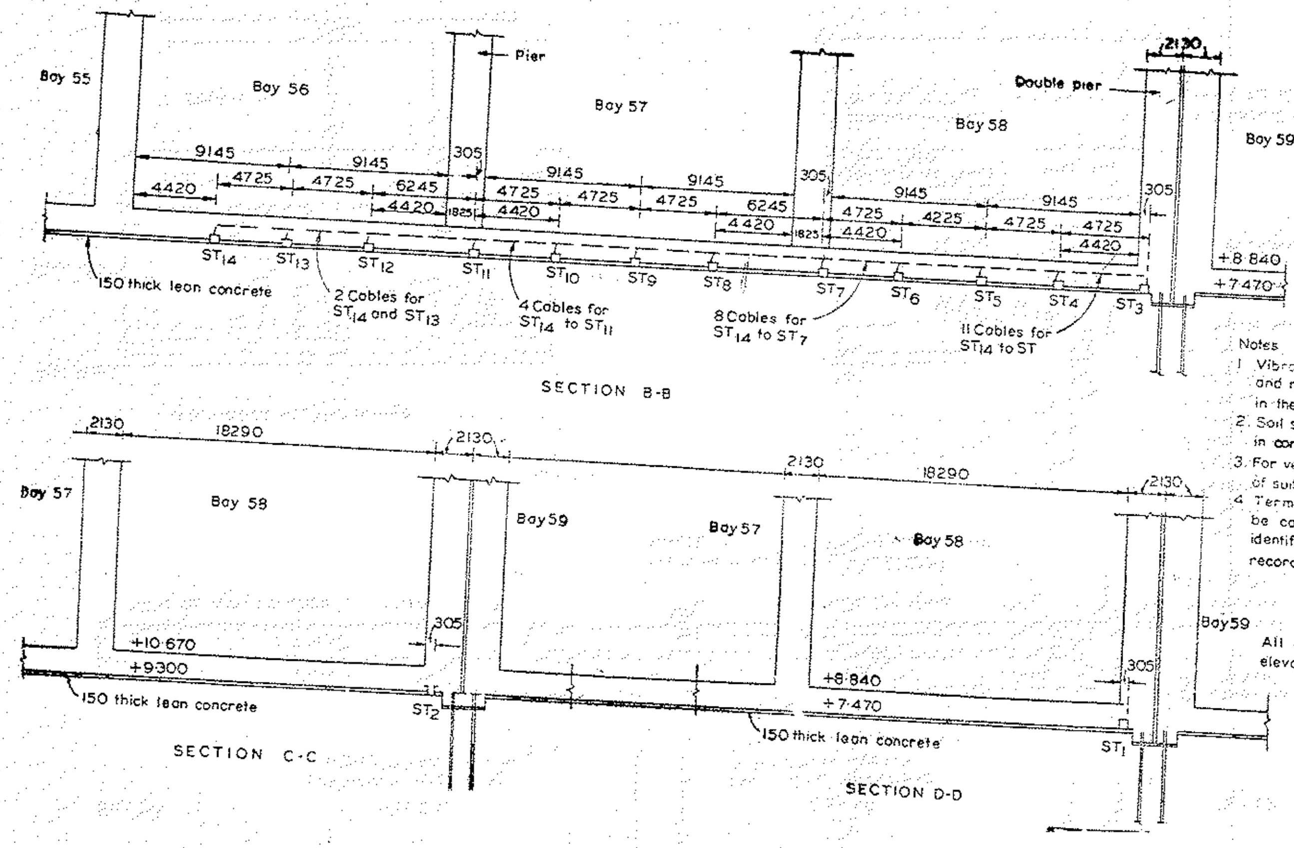


FIGURE 8.4: Typical Details of Location of Instruments and Recording Point.



Notes

1. Vibration wire type instruments and relevant cables are indicated in the figure
2. Soil stress meters are to be laid in contact with foundation soil
3. For vertical run of cables conduits of suitable size are to be provided
4. Terminal of each instrument should be carefully marked for easy identification and initial reading recorded for each instrument

All dimensions are in mm and elevations in metres

FIGURE 8.5 : Typical Details of Location of Instruments and Recording Point,

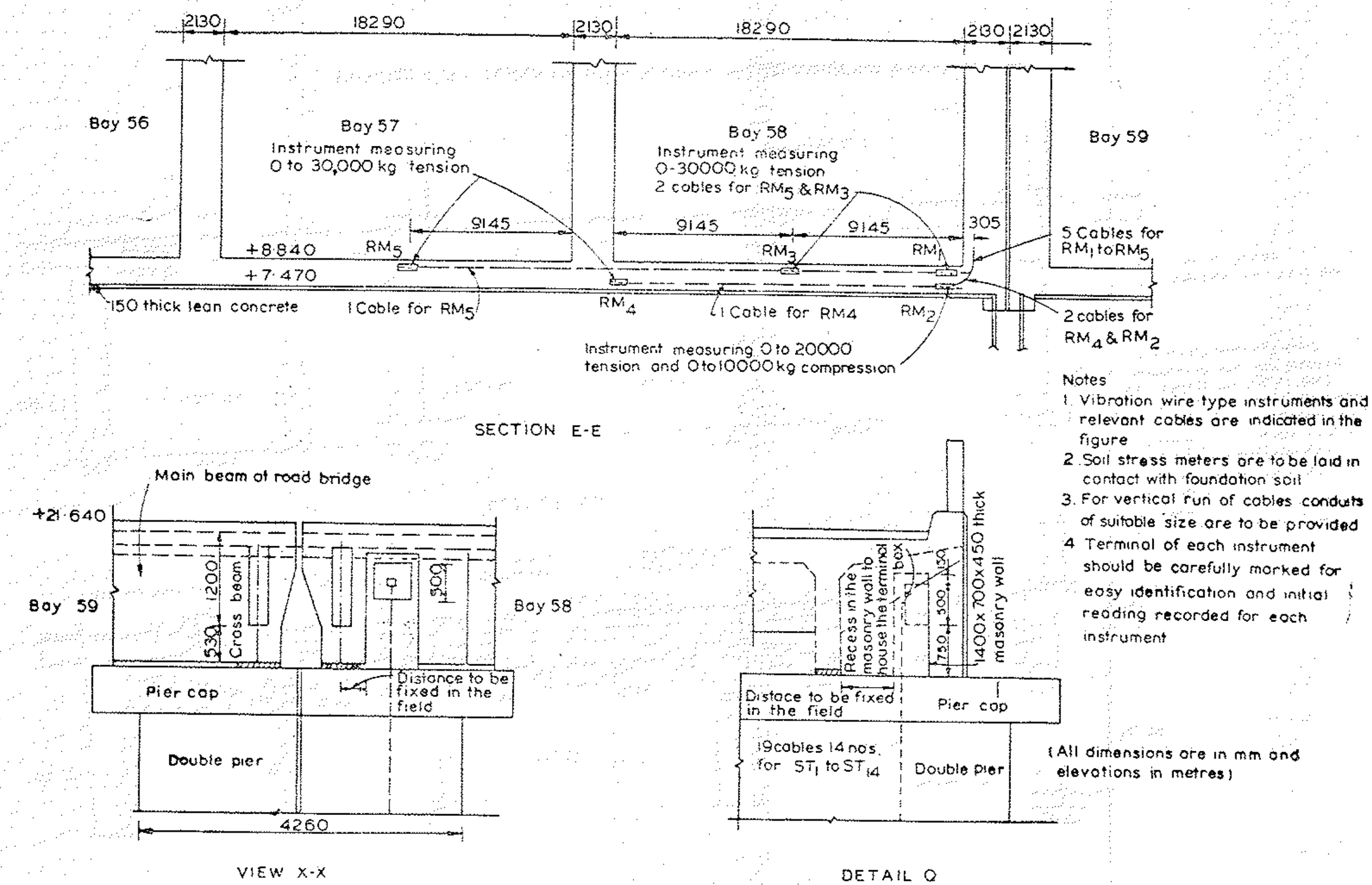


FIGURE 8.6 : Typical Details Location of Instruments and Recording Point.

CHAPTER 9

Construction

9.0. Introduction

Diversion structures are light weight structures compared to storage dams and are designed taking some calculated risks. Hence it is very important that the constructions of the same are carried out with great care and there is no room for construction failure to occur. River diversion works for taking up the construction and the various construction operations are not proposed to be discussed in this Chapter and are left to the planning of the constructing agency. Reference can also be made to relevant codes and text books for the same. In the construction of the different components of the diversion structure, certain points are to be properly taken care of. Only these are discussed in this Chapter.

9.1. Co-ordination between the Designing and Constructing Agencies

A good co-ordination between the Designing and Constructing Agencies has to be established for bringing out a safe and economical structure. The constructing officer should be in the knowledge of the implicatons of the provisions in the designs of the various components. Any change required to be made on account of site conditions or any other reason whatsoever should not be attempted by him on his own without evaluating the implications due to such changes and the designer should invariably be consulted to find out any other economical and safe alternative thereof. Frequent checks on the construction vis-a-vis design provisions and constant quality control must be ensured.

9.2. Construction Data

The following construction data should be collected for carrying out the construction efficiently :

(*i*) Sequence of construction of various items of the structure

(*ii*) Various constraints on different activities of construction of the components.

(*iii*) Inter-dependence of various items so that there is minimum interference in the continuity of progress.

(*iv*) Obligatory precautions to be taken for protection of season's works from ensuing floods and

(*v*) Special features, if any.

9.3. Foundations

Foundation preparation is an important item since the whole structure is going to rest-on that. It has to be dressed up to the barrage/weir profile and excavation has to be done carefully without exceeding the tolerance limits. The foundation should not contain loose pockets or materials and they should be watered and compacted to the specified relative density. Clay pockets should be treated as specified by the designer. It has to be ensured that proper drainage arrangements in the foundation as per the designs including inverted filter wherever indicated are provided before concreting work is taken up. Wherever instruments are to be provided in the foundation as per designs, they should be located carefully with all precautions. To avoid honeycombing of concrete of the floor, proper mudmat of lean concrete usually 50 to 150 mm thick should be provided

9.4. Cut-Off

Wherever sheet piles are to be provided, they must be driven at their correct alignment without any gaps between them in the wall Since it may not always be possible to drive them plumb suitable tolerances may be allowed. If the tolerances exceed the permissible limit, they should be corrected by Taper piles. If there is any split in the interlock additional piles in front should be driven to cover it. Making of piles either by welding prior to driving or welding in-situ to make up the height is normally adopted. Driving of welded sheet piles of more than 8 m length is not safe as weld may give way due to excessive driving stresses. Wherever seals are provided and are to be joined to the sheet piles. it should be carefully done by welding and brazing. Since sheet piles are designed to have hinge action at the top, it should be ensured by provision of tar paper wherever two pile rows are provided side by side, cork mastic filter on top may be provided to take care of uneven heights & hinge action. Wherever concrete/masonry cut-offs are provided, precautions must be taken to avoid cracking as it may lead to short circuiting of seepage path.

9.5. Pucca Floor

The pucca floor has to be constructed carefully avoiding stratification of concrete which may lead to failure by blowing off against uplift pressures. Cold joints should be avoided. The strength of cement concrete of the raft must be maintained as per specifications so that the stresses in both the concrete and steel reinforcement are not exceeded. The main and distribution reinforcement of the raft must be carefully laid as per specifications and design requirements. Reinforcement around sill beam grooves should not be omitted. The raft reinforcement would be in both top and bottom of the raft. The required distribution reinforcement at specified spacings would be provided at the top of bottom reinforcement and bottom of top reinforcement. The pier reinforcement would have to be properly anchored to the raft.

9.6. Pier and Gates

In the piers, correct alignment of the gate and stoplog grooves should be ensured so that no difficulty is experienced in their operation, especially during floods. For inspection and repairs of the gate wheels, wherever contemplated, suitable galleries in the piers with easy access should be provided. The pier reinforcement projecting out in the case of unfinished piers should be suitably protected by bending so that they do not snap during floods. While concreting, the pressure relief pipes installed in the piers and their outlets should not be lost sight of. The pipes required for instrumentation should also be fixed before hand through the piers. In the case of gravity type of floor, the stepped pier footings should be concreted upto the barrage/weir floor level and further portion of pier above the bay level should be concreted simultaneously with the bay concrete. The steps should be so provided that there is no vertical joint. In the raft type of floor, the bay concreting and pier concreting should be done simultaneously.

9.7. Abutment

Anchorage of the abutment reinforcement to the base slab must be ensured. The abutment wall should be raised simultaneously along with the backfill and in any case should not be more than about 1.5 m above the compacted backfill. Specified relative density of the backfill must be ensured by proper compaction. Wherever drainage is to be provided behind the abutment walls as per designs, the same must be done carefully so that the saturated water level of the backfill is not allowed to exceed the designed values. Inverted filter and sealing arrangements at the junctions of the different abutment blocks must be properly ensured.

9.8. Return Wall

If the return wall is constructed at the ends of the abutment, it should be properly keyed to the bank. If the return wall is forming a part of the flank wall, while it is monolithic with the base slab, a joint with seal should be provided between the stems of the flank wall and return wall. The reinforcement of the base slab should be carefully laid out as per designs.

9.9. Divide Wall

As the divide wall comes under direct attack of the flood flow, proper protection around the same as per designs must be provided. The change in the levels of the bed in front of the sluice and spillway portion must be gradually made up and abrupt changes avoided. The junction of well cap of divide wall and the bay concrete should be done with proper precautions.

9.10. Downstream Protection

As the downstream inverted filter below the cement concrete blocks is very important as a measure against piping, it has to be laid with due care. The gaps (jharries) between the c.c. blocks should also be filled with pervious peastones or bajri. Wherever the downstream bed level is higher then the level of downstream c.c. block protection, reverse slopes, not steeper than 1 in 5 may be provided and it may be ensured that some loose stone protection is provided for a length of not less than about 2 m in the higher bed portion after the reverse slope. Wherever precast c.c. blocks are used for protection works, precast yard of suitable size should be provided for casting and stacking of precast blocks. For handling of the precast blocks, cranes/gantries of suitable capacity should be used. In case of cast-in-situ blocks, alternate blocks should be concreted or a proper pattern of concreting sequence should be established for economical use of block shuttering. The formwork should be so designed that when it is stripped off, the required gap is formed for filling in filter material. Cast-in-situ blocks on slopes of guide bund or flare out walls should be cast with due care.

9.11. Instrumentation

The importance of instrumentation must be understood fully and care must be taken in their installation so that wrong data are not observed leading to misleading and dangerous conclusions. The instrumentation observation panel must be located at convenient places and easy access to it provided so that recording the data at site does not became a bottleneck.

9.12. River Training Works

The materials used in the construction of various river training works such as guide bunds, afflux bunds, approach embankments, groynes or spurs, etc., and their construction itself should be of required standards so that their failures do not occur impairing with the hydraulic performance of the diversion structure. Wherever filters are indicated in the designs, they must be provided without fail since the stability of the bunds would depend on it. It must be ensured that the afflux bunds are tied to high grounds to prevent outflanking of the structure.

9.13. Head Regulator

Head regulator is also a structure similar to the main structure so far as the general principles of designs are concerned. Hence whatever precautions are to be observed in the construction of the main structure, they should be followed for head regulators also.

9.14. Formwork

The formwork should preferably be in steel except for curved surfaces, block outs, etc., where timber/plywood formwork could be adopted. For repetitive use like in piers, the formwork should be of firm construction. The main criterion for forms should be the functional requirements. For exposed surfaces, non-staining form oil should be used and for buried surfaces, the waste-oil available at site may be made use of.

9.15. Reinforcement

Chair supports, spacers, etc., should be provided as a part of reinforcement for proper positioning of reinforcement. In case of raft foundation having two layers and reinforcement the bottom layer would be laid first and concreting done upto appropriate levels. Chairs, supports are to be embedded in the last lift. The top reinforcement would then be laid and concreting done. Proper lapping and bending of reinforcement, wherever necessary, must ensured.

9.16. Joints

For all construction joints, the normal methods of clean-up and surface treatment should be adopted. In case of rubber seals, specified formworks should be provided to support the rubber seals. Concrete in the vicinity of waterstops should have a slump of 75 to 103 mm to facilitate easy placement.

9.17. Architecture

Wherever possible the shape of the pier ends, finish of the abutment, pier, divide wall surfaces, etc., can be modified to increase the architectural beauty of the structure. But hydraulic performance and safety should not be sacrificed.

CHAPTER 10

Maintenance and Operation

10.0. Introduction

Proper inspection, maintenance and operation of the diversion structures are necessary adjuncts to safe and economical designs. Any slackness in these aspects would lead to failures and extension of damages. It is also necessary that a history of the diversion structure including the damages, repairs, maintenance and operation are kept available so that the maintenance and operation could be reviewed and modified wherever necessary. These aspects are discussed in brief in this Chapter.

10.1. Inspection and Maintenance of Civil Works

Regular and careful inspection and maintenance of civil works must be carried out for all the components, both under water and above. Necessary repairs should be carried out in time before damages, if any, are extended. These are detailed below :

10.1.1. *Upstream and Downstream Aprons* : After the monsoon, sounding and probing should be done both on the upstream and downstream sides. These would be carried out by sounding rods or echo sounders over the c.c. block and loose stone protections. Contour maps and sections could be prepared to determine the places of scour, its extent, launching, etc. The effectiveness of the downstream inverted filter could also be determined. Wherever scours have taken place extensively, the apron could be replenished to the designed values.

10.1.2. *Pucca Floor* : The pucca floor, both upstream and downstream should be thoroughly inspected for cracks, wear and tear, cavitation. etc. Energy dissipating devices like chute blocks, friction blocks, and sills etc. should also be checked for damages. While dewatering deep downstream basins for inspection, it should be ensured that the design uplift values for the pond level condition are not exceeded. Necessary repairs should be carried out immediately.

10.1.3. *Sediment Excluding Devices* : Sediment excluder tunnels and the deflectors should be thoroughly inspected for cracks, choking, etc. with the help of divers and underwater lamps. Wherever necessary, desilting of chocked tunnels and channels should be carried out and other repairs attended to.

10.1.4. *Piers* : The piers should be inspected carefully for settlement, cracks, tilting etc. The noses of the piers should be checked for damages if any due to boulders, floating debris, etc. Necessary repairs must be attended to immediately for proper functioning of the gates, stoplogs, bridges, etc.

10.1.5. *Abutments* : Inspection of abutments should be carried out similar to the piers. In addition, the backfill should also be examined for settlement and if need be, it should be made up and properly compacted.

10.1.6. *Flankwalls* : Inspection and maintenance of flankwalls are similar to those for the abutments. Damaged c.c. blocks of the flared out walls, if any, should be replaced so that hydraulic performance of the structure is not impaired.

10.1.7. *Divide Walls* : In addition to the inspection like that for the piers. the effectiveness of the drive walls for satisfactory hydraulic performance should also be observed and necessary corrective measures adopted. The protection works around the noses and shanks of the divide walls should be checked up by sounding and replenished to design values wherever necessary.

10.1.8. *Bridges* : The beams, slabs and the wearing coats of the bridges should all be checked for cracks, wear and tear. joints, etc. and repairs carried out so that smooth traffic is ensured.

10.1.9. *Fishpass* : Apart from the usual inspection necessary for the concrete structure of the fish pass, its effectiveness for the smooth movement of fish from upstream to downstream and vice versa should be studied and corrective measures adopted wherever necessary.

10.1.10. *Navigation Lock* : The concrete structures of the navigation lock should be thoroughly inspected for cracks, settlement, etc. and repairs carried out. If the lock chamber and also the navigation channel are silted up, necessary desilting operations should be carried out. It must be ensured that the filling and emptying times are able to be maintained as per design values. Any difficulties in the same should be removed so that free flow of navigation is maintained Approaches to both the upstream and downstream navigation locks should be maintained properly to avoid any congestion and damages.

10.1.11. *Head Regulators* : The inspection and maintenance of the head regulator are similar to those of the main barrage/weir. The discharging capacity of the head regulator should be measured and if difficulties are experienced in passing the required discharge, necessary corrective measures should be adopted, if need be, through model studies.

10.1.12. *Instruments* : It is necessary that every year, performance reports are prepared, based on the different instrument observations. The observations could be broadly classified under (*i*) Uplift pressure (*ii*) Suspended sediment (*iii*) Settlement (*iv*) Retrogression (*v*) Upstream aggradation and (*vi*) Discharge distribution and cross-flow.

10.1.12.1. UPLIFT PRESSURE

The details of uplift pressure pipes, their installation and observation may be seen in the chapter on Instrumentation. It may be adequate to take observations once a month during monsoon period and more frequently during the non-monsoon period. However, it should be ensured that

(i) the mouths of all pipes are kept closed by caps to avoid the chances of extraneous materials getting into the pipes and clogging them.

(*ii*) the pipes are properly and clearly numbered for identification

(*iii*) the pipes are frequently tested to make sure that the strainers are not choked.

The quantity and quality of the sediment content of the water coming out of pressure release pipes on the downstream floor should be tested during dry season. This would indicate their efficiency and also the event of undermining of the foundations, so that necessary remedial measures can be adopted on time.

10.1.12.2. SUSPENDED SEDIMENT

The silt charge of water upstream and downstream of the undersluices and in the canal below the head regulator should be observed frequently so that the efficiency of the silt excluding devices can be assessed and the gate regulation can be modified suitably to improve the efficiency.

10.1.12.3. RETROGRESSION

It is necessary to measure the retrogression of the downstream river bed so that if the limits over the assumed values are exceeded, necessary remedial measures can be taken on time to ensure the safety of the structure.

10.1.12.4. UPSTREAM AGGRADATION

Upstream aggradation of the river bed increases the afflux and consequent encroachment into the free board of the components of the diversion structure including the river training works. Hence it is necessary to know the aggradation taking place so that the top levels of the components can be raised wherever necessary.

10.1.12.5. DISCHARGE DISTRIBUTION AND CROSS FLOW

Designs are carried out assuming certain concentration of flow through the different bays of the diversion structure. Hence it is necessary to know the discharge distribution through the bays so that the designs could be checked up for the safety of the structure. Similarly crossflows should also be observed and proper remedial measures taken for ensuring the saftey of the structure. This could be by suitable modification of the gate regulation pattern.

10.1.13. *River Training Works* : It is necessary to have maps of the river showing the configuration of the river every year so that the effectiveness of the river training works could be studied and modified if necessary. The points of attack by the flowing water should be observed and strengthened. For this purpose, a good stock of loose stone should be kept near vulnerable spots.

10.2. Inspection and Maintenance of Mechanical and Electrical Works

In addition to the inspection and maintenance of the civil works, the mechanical and electrical works also should be inspected and maintained regularly. If they are not kept clean, tidy and in proper working order, they would fail at the time of emergency leading to damages of the entire structure. Under the mechanical and electrical works the following could be included :- (*i*) Gates and falling shutters (*ii*) Gate grooves and seals (*iii*) Steel wire ropes (*iv*) Roller trains and fixed rollers (*v*) Winches/hoist (*vi*) Flood lighting (*vii*) Bridge bearings and Super structures.

10.2.1. *Gates and Falling Shutters* : The gates and falling shutters should be kept clear of the debris and silt accumulations and they should not be allowed to rust due to improper drainage. The upstream face of the skin plate coming into contact with water should preferably be painted with suitable primer and subsequently with sanded aluminium paint for long life. While painting the surfaces, all necessary precautions should be observed.

10.2.2. *Gate Grooves and Seals* : Gate grooves and particularly their machined faces should be kept clean and lubricated well and all their sticky deposits should be scraped off before applying the lubricant. The efficiency of the rubber seals should be tested and also examined for wear and tear and deterioration. Their replacement whenever necessary should be done immediately.

10.2.3. *Steel Wire Ropes* : All steel wire ropes should be cleaned to remove all the dust accumulation and lubricated with suitable greases atleast once a year. The portion of the wire ropes submerged in water, after lubrication, should be wrapped with gunny bags which should be securely fastened to the ropes. The clamping devices also should be inspected for their efficacy.

10.2.4. *Roller Trains and Fixed Rollers* : The roller trains and fixed rollers should be cleaned, free movement ensured and greased for smooth operation worn out rollers and pins should be replaced and bolts etc., tightened.

10.2.5. *Winchcs/Hoits* : Winches and lifting drums should be kept clean and greased properly for smooth and easy operation. Alignment of shafts should be checked and coupling bolts tightened. In the case of electrically operated hoists, all the electrical wirings, switches, bearings, reducing gears, etc., should be checked for a safe and trouble free operation. The platform should also be examined and properly protected.

10.2.6. *Flood Lighting* : During the flood season all flood lighting and site illumination should be checked daily and during slack season, it may be done once in a week.

10.2.7. *Bridge Bearings and Super Structures* : The bridge bearings over the piers and abutments should be cleaned and greased once in a year after the monsoon season. The painting of super structures should be done once in two years.

10.3. Operation and Regulation

The operation and regulation of the diversion structure can be divided into three periods, viz, (*i*) Pre-monsoon (*ii*) During monsoon and (*iii*) Post-monsoon. Detailed instructions in this regard issued by higher authorities should be followed.

10.3.1. *Pre-monsoon Operation* : No wastage of water can be permitted during this low-flow period and the gates/falling shutters should be judiciously regulated to maintain the pond level. The discharge thro' the head regulators should be regulated according to the approved discharge tables to feed the canals.

10.3.2. *Monsoon Operation* : During monsoon period, the operation of the gates and shutters should generally follow the recommendations of the model studies, as modified by the observed river behaviour at site. Operation has to be done judiciously to improve flow conditions and avoid shoal formations. The pond level should be kept minimum to feed the canal with the required discharge and excess passed through the diversion structure. In high floods, all gates should be raised clear of the HFL with sufficient margin for floating debris to pass through and all falling shutters must be lowered. Whenever the sediment charge of the flood waters increases beyond the permissible limits for the off-taking canal, the head regulator gates should be closed. Under-sluice gates should not be allowed to be overtopped and sediment excluders should be operated to prevent silt entry into the canal as much as possible.

10.3.3. *Post-monsoon Operation* : Still/semi-still pond operation, with the silt excluding/extracting devices operating, should be continued till water becomes reasonably clear. After the soundings are completed and the river bed levels are known, suitable regulation should be evolved so that shoals are eliminated as much as possible.

10.3.4. *Operation of Gates and Falling Shutters* : The gates and falling shutters should be operated according to certain guide lines indicated by the model studies, general. observations, design calculations, etc. However,, certain precautions should be observed while operating them.

10.3.4.1. All lift gates should be operated at suitable interval to free the mechanism and wash out extraneous material. In low supplies, the gates could be raised by about 15 cm for a few minutes. If the gates have not been lifted up for a long time, they should be lifted by about 3 cm or so and kept in that position for about 10 to 15 minutes till the silt deposited against the gates gets softened. This would avoid heavy strain on the machinery.

10.3.4.2. The maximum speed of operation of the gates specified by the manufacturers should not be exceeded.

10.3.4.3. The head regulator gates should generally have equal openings unless otherwise indicated by model tests.

10.3.4.4. When the gate operation is of wedge type, the gates should be opened in instalments not exceeding 30 cms at a time. The gate opening should be suitably increased to allow passage of boulders.

10.3.4.5. While closing the gates operating the silt excluder tunnels. it has to be done very slowly to avoid water hammer action leading to damages to the structure.

10.3.4.6. When a canal is first opened, low supply should be run for a few hours at least and depth should be gradually raised according to the requirements. The rate of filling and lowering of the canal water level should not be violated beyond the prescribed limits.

10.4. Record of History of Headworks

It is always desirable to keep an up-to-date history of the headworks including the maintenance, damages and repairs carried out from time to time, schedule, of gate operation prescribed and actually carried out with reasons thereof, behaviour of the river, etc. This would help in evolving a modified gate operation schedule wherever necessary for better hydraulic performance and safety of the structure.

CHAPTER 11

Effect of Diversion Structure on the Regime of the River

11.0. Introduction

Before proceeding with the actual designs of the diversion structure, it would be desirable and helpful to study how the river would be affected by the construction of the barrage/weir across it since its natural behaviour is going to be interfered with. This study would indicate what precautions are necessary to be incorporated in the designs to accommodate the anticipated changes in the regime. These are discussed in this Chapter.

11.1. Accretion and Retrogression of River Bed

The first effect of the construction of the diversion structure is a flattening of the water surface slope for some distance behind the structure due to ponding up of river supplies. This results in reduction of sediment transporting power in that reach so that the river drops a part of its sediment load resulting in the formation of shoals and islands in the pond. The relatively clearer water passing over the structure picks up sediment from the river bed to make up the deficit in its content and causes a progressive lowering, otherwise called retrogression, of downstream levels.

The retrogression process continues for a number of years. Due to the increase in the shoal formation in the upstream reaches, the resistance to flow of the river increases as the route of the river flow becomes tortuous around the shoals. To overcome this resistance, increased head is required and the river starts to regain its original slope. The afflux on the upstream thus gets extended. A stage would arise when the upstream pond does not absorb any more silt. Due to the silt excluding devices provided and also a higher crest elevation of the head regulator, comparatively less silty water is drawn through it and the silt burden would go downstream whereas there would be discharge below normal on the downstream. This would lead to silt deposit on the downstream and a long range recovery of levels. These in short are the effects of the construction of a diversion structure on the regime of the river.

11.2. Effects on the New Design

Precautions in the designs are to be taken as mentioned below due to the anticipated changes in the river regime.

11.2.1. The high flood levels on the downstream side would be lowered due to retrogression which is generally taken as 0.3 to 0.5m for higher discharges and 1.25 to 2.25 m for lower design flood. The retrogressed Stage Discharge curve thus prepared would have to be used in the designs for energy dissipation and exit gradients.

11.2.2. Restoration of normal slope upstream of the diversion structure leads to increased flood levels by the extent of afflux on the upstream over the original back water curve. When it is observed that due to accretion of the upstream bed, the free boards of the various river training work are encroached upon, their levels should be raised suitably and there might be needs of extension of afflux bunds also.

11.2.3. Recovery of downstream bed levels sometimes may continue even beyonds the original bed levels. This may lead to loss of control on silt regulation. Apart from the increase in the top levels of the downstream components to take care of the downstream high flood levels, sufficient margin would need to be provided between the Full Supply Level of the canal and the pond level, to provide for the required cut-off at the head regulator for passing the discharge. It should also be possible to raise the crest level of the head regulator at a future date if necessary.

CHAPTER 12

Failures of Barrages and Weirs

12.0. Introduction

Diversion structures, barrages or weirs, are essentially light structures whether they are of gravity type or raft type. Generally they are founded on permeable soils though there are quite a few structures founded on rocks. These diversion structures have been and are being built by taking calculated risks. Any undue occurrance of severe floods/flash floods or a violent change in the direction of flow and sometimes aided by faulty construction may affect the safety of the structure. Many structures have failed in the past and many lessons have been learnt from these failures. In fact the modern designs of diversion structures have been developed from the analysis of these failures. In this Chapter, the causes for failure of the diversion structures and remedial measures are discussed. The various points to be looked into and data to be collected in the event of a failure of a barrage or weir are also indicated in this Chapter.

12.1. Causes of Failure

The failures of the diversion structures can be attributed to the following main causes, acting singly or in combination :

(*i*) Undermining through piping due to excessive exit gradient.

(*ii*) Eruption of floor caused by uplift exceeding gravity forces.

(*iii*) Deep scour in the immediate vicinity upstream or downstream of the pucca floor.

(*iv*) Faulty regulation.

(*v*) Faulty construction.

(*vi*) Severe flood leading to overtopping and/or out-flanking.

These are further discussed in subsequent paras.

12.2. Undermining through piping due to excessive exit gradient :

The importance of obtaining a safe exit gradient for the diversion structure has already been discussed in Para 6.1.5.4. (Vol. I). In Chapter 6 on Hydraulic Designs. The safe values according to Bligh's Theory, Lane's Weighted Creep Theory and Khosla's Theory have also been indicated in the Chapter for various types of soils. Excessive exit gradients cause piping below the floor of the structure which leads to subsidence and ultimate collapse. Piping is caused by the absence of adequate depth of downstream cut-off, absence and/or non-functioning of inverted filter on the downstream of the floor, short-circuiting of seepage flow carrying the foundation soil through the cracks and cavities in the floor, absence of cross-cut-offs below the divide walls separating the undersluices and spillways thereby short-circuiting the seepage path, improper sealing arrangements in the various joints provided, especially in the floor, etc.

12.2.1. Such a failure and damage has occurred in a number of weirs and barrages in the past. Notable among them are Deoha Barrage (1929), Khanki weir on Chenab river (1932), Anderson weir on Damodar river (1935), Old Menugia Regulator in Egypt, Islam weir on Sutlej river (1929), Marala weir on Chenab river (1934), Sarda Barrage on Sarda river (1956) and Mississippi weirs (USA).

12.2.2. In the event of a failure due to excessive exit gradient, the remedial measures after repairing the damages would be to provide a deeper cut-off on the downstream and provision of properly designed inverted filter below the c.c. blocks on the downstream. In case it is not possible to provide a deeper downstream cut-off due to any reason whatsoever, the floor length can be increased on the upstream side subject to economical considerations as discussed in para. 6.8.(Vol.I). However, it has to be ensured that there is no short-circuiting of seepage path rendering the increased length ineffective in reducing the exist gradient.

12.3. Eruption of Floor Caused by Uplift Exceeding Gravity

The failure due to eruption of floor caused by uplift exceeding gravity can occur when there is no water running downstream and the pond is maintained on the upstream, or when water is flowing downstream through the gates and hydraulic jump is formed over the glacis. The failure occurs when adequate thickness of the floor is not provided in the trough portion. The floor may contain lot of cavities inside rendering it ineffective to resist the uplift pressure. Excessive uplift pressures occur due to inadequate drainage arrangements provided below the floor or any locked up pressure due to clay pockets or the downstream cut off embedded in to an impermeable layer. The extent of uplift pressures developed below the floor of major structures can be assessed if proper instrumentation has been installed and the data analysed.

12.3.1. Failures due to eruption of floor caused by uplift exceeding gravity have occurred in Narora weir (1898), Marala weir (1934), Rasul weir on Chenab river (1929), Deoha Barrage (1929), Khanki weir (1932) and Sarda Barrage (1956).

12.3.2. Remedial measures for such failure consist in increasing the thickness of the floor with due considerations for adequate cistern level for proper energy dissipation, provision of reinforced cement concrete floor raft instead of gravity section, grouting of cavities formed if any and increase of upstream floor so that the percentage pressures at key points especially over the trough region can be reduced by increasing the seepage path.

12.4. Formation of Deep Scours

There are occasions when deep scours are formed either or both on the upstream and downstream of the floor. Failure occurs if cut offs both on the upstream and downstream are not provided for adequate depths. Deep scours occur caused by high intensities of discharges through the structure. This may be due to inadequate waterway or faulty gate regulation. If proper flexible protections are not provided, the foundation soil is removed by deep scours and consequently the structure fails. Scours are also formed due to improper energy dissipation. If retrogression on the downstream of the bed has not been accounted for, hydraulic jump may form outside the floor and with no proper apron provided, deep scours occur leading to failure.

12.4.1. Failures due to improper energy dissipation and formation of deep scours have occurred in Islam weir on Sutlej river (1929), Khanki weir (1932), Anderson weir (1935), Marala weir (1934), Sarda Barrage (1956) and some Mississippi weirs (USA).

12.4.2. When such failure occurs, it would be necessary to lower the cistern level and provide adequate energy dissipating arrangements such as end sill, dentated sill, friction blocks, chute blocks, etc. To reduce the flood velocities, the upstream apron level can be lowered. The depths of upstream and downstream cut-offs should be properly calculated with safe scour factors and provided for. The upstream and downstream aprons should be properly replenished to the designed values. Steeper glacis slopes may need to be flattened to say 3 : 1.

12.5. Faulty Regulation

Sometimes failures have occurred due to faulty regulation of gates, resulting in high concentration of flow through the structure. When such a thing occurs, inevitably the energy dissipation is not proper and deep scour occurs. Sometimes it may so happen that damages occur due to one or other causes mentioned in para 12.1, but the gates might have been lowered giving priority to feeding the canals which results ultimately in major damages, as happened in the case of Islam weir on Sutlej river in 1929. Faulty regulation can also be traced to poor maintenance of gates, because of which they get jammed and are not able to be operated when desprately needed.

12.5.1. Failure due to faulty gate regulation can be prevented only by imparting adequate knowledge to the operating personnel about the significance of proper regulation and proper maintenance of gates and other hoisting arrangements.

12.6. Faulty Construction

Failures of diversion structures due to faulty construction are not uncommon. Faulty construction also leads to other causes of failures such as piping below the floor, eruptions of floor due to uplift pressure, faulty regulation, etc. Various items coming under the category of faulty construction can be (*i*) stratification of concrete layers in the pucca floor and no proper bond between the layers of concrete (*ii*) various construction and structural design defects (*iii*) cracks in the downstream glacis leading to short-circuiting of seepage path (*iv*) improper foundation treatment leading to subsidence and consequent disturbance in the alignment of gate track and jamming (*v*) cracking of seals in the joints (*vi*) tearing of sheet piles and improper interlocking (*vii*) improper foundation treatment of impervious layers leading to locked up seepage pressure (*viii*) inadequate cover for reinforcement bars (*ix*) honey-combing of bottom layers of floor concrete due to omission/inadequate mudmat (x) improper concrete mix used resulting in structural failure, etc.

12.6.1. In the recent past, failures due to faulty construction have occurred in Islam weir (1929), Rasul weir (1929), Anderson weir (1935), Marala weir (1934) etc. due to one reason or the other.

12.6.2. To avoid failures due to faulty construction, strict quality control and supervision of construction should be ensured. Safety should not be compromised for cost or speed of construction. To remedy the failures, the spots where faulty construction have occurred must be identified and suitable measures taken.

12.7. Overtopping

Failures due to overtopping of the components of the diversion structure occur mainly due to the wrong estimation/adoption of the design flood and maximum flood for free board purposes. This results in more afflux than the estimated one and with the provision of less free board, overtopping naturally occurs leading to damages. There are quite a few instances where due to inadequate free board, overtopping of guide bunds has occurred and this coupled with faulty construction leads to collapse of guide bunds and further damages. When the top of the piers or abutments are not fixed properly, due to overtopping, damages to the bearings of road/rail bridges occur leading to failures.

12.7.1. Remedial measures or rather preventive measures to avoid failures due to overtopping consist in the correct assessment of the design flood for waterway and maximum flood for free board purposes and provision of adequate free boards all along taking the siltation aspect also into account.

12.8. Outflanking

Failures due to outflanking are related to the wrong estimation of the flood values and subsquent overtopping of afflux bunds, tie bunds, etc. Another reason could be placed on the wrong location of the diversion structure in geologically weak spots, especially the banks. On highly meandering rivers, if marginal embankments and jacketting of the rivers are not done properly, failures due to outflanking cannot be ruled out. Improper gate operation also leads to outflanking aud failures thereof.

12.9. Record of Failures and Remedial Measures

For each and every diversion structure, either a barrage or a weirs, it is desirable to have a record of its behaviour, both hydraulic and structural. The record should also contain the details of its failures if any and remedial measures adopted from time to time. Necessary photos should also be available of the same. These would always help for future modifications, if any, needed for the structure and also for others.

12.10. Data to be Collected in the Event of a Failure

Whenever there is any failure of the diversion structure, quite a few data have to be collected for study and analysis so that suitable remedial measures can be adopted. A list of such data is given below :

(*i*) Detailed drawings of the diversion structure, regulators, guide bunds, afflux bunds, approach bunds, spurs, etc.

(*ii*) Detailed note on the damages noticed including history and remedial measures carried out if any so far.

(*iii*) Development of the damages.

(*iv*) Photographs taken if any.

(*v*) Discharge and water level values at various points along guide bunds, afflux bunds, approach bunds, spurs, abutments, etc.

(*vi*) Flow pattern observed i.e., concentration of flow through some bays or otherwise.

(*vii*) Sounding data both upstream and downstream at the ends of the raft, cement concrete blocks, stone protection, around divide walls and for a distance of 60m or so at 15m intervals.

(*viii*) Gate operation followed during floods and other times.

(*ix*) Any seismicity experienced prior to or during the floods.

(*x*) Data on settlement of piers, abutments, flankwalls, divide walls, etc., if any.

(*xi*) Data on tilting of piers, divide walls, abutments, flankwalls etc., if any.

(*xii*) Quantity and quality of bed materials and floating debris during the floods.

(*xiii*) Details of spurs, upstream and downstream.

(*xiv*) Data on traffic over the structure if any during the floods.

(*xv*) Details of construction materials used including quality of construction for various components including the different bunds.

(*xvi*) Instrumentation data available if any.

(*xvii*) Details about stages of construction.

(*xviii*) Details of any field investigations done for detecting hollows if any and results thereof.

(*xix*) Previous history of any damages and remedial measures there of.

(*xx*) Recommendations of any Technical Advisory Committee from time to time and compliance.

(*xxi*) Hydraulic and structural designs with assumptions made if any.

(*xxii*) Any other data relevant to the case under investigation.

12.11. After working out the necessary remedial measures and implementing them, it is desirable to publish the causes of failures and remedial measures adopted in an article for the benefit of designers and project authorities.

CHAPTER 13

Some Special Problems

13.0. Introduction

In construction, maintenance and operation of barrages and weirs, in addition to the usual problems of siltation, scouring, etc., sometimes some special problems are encountered depending on the location where the structure is constructed and precautions are taken during construction. Some of these problems could be (*i*) ice problem (*ii*) damages due to rolling stones (*iii*) cracking of concrete (*iv*) transport of logs (*v*) shearing off of pier reinforcement and (*iv*) special seepage problems. These are discussed here in this Chapter. The list is not exhaustive, The other problems have to be tackled with all due considerations for safety and economy.

13.1. Ice Problems

Diversion structures constructed in high altitudes face the problem of formation of ice over the structures especially the moving parts like gates, etc. Ice forming over the water surface may exert extra pressures over the abutments, piers, divide walls, etc. The components affected should be checked for stresses due to the ice thrust. To prevent the ice formation over the moving parts, usually a heating system for the gates is provided and sometimes, air bubbler system is also provided. A typical gate heating arragements is shown in Figure 13.1.

13.2. Damages due to Rolling Stones

Diversion structures located in bouldery reaches face this problem. The rolling stones may damage the nose of the piers or the pucca floor. The top thickness of the floor is made of richer concrete to resist abrasion. Sett stones are also provided at the top to resist abrasion. Experiments with application of epoxy resins over the surface are still in progress. The nose of the piers are usually provided with steel cladding. A typical steel cladding arrangement is shown in Figure 13.2.

13.3. Cracking of Concrete

Due to extreme variations in temperature, there could be thermal cracks in the concrete of the floor, piers, etc. Cracks may also be caused due to differential settlement. The exact nature of the cracks would have to be ascertained and remedial measures adopted. Epoxy grouting of cracks is one of the methods usually employed for this purpose. In case of differential settlement, necessary foundation treatment will have to be resorted to.

13.4. Transport of Logs

For the transport of considerable amount of timber from upstream of a diversion structure to the downstream, provision of a log chute may be necessary. However, the economics of having a log chute vis-a-vis mechanical handling would need to be worked out. In the case of weirs, the crest is depressed by about 0.5 to 1.[illegible] m and provided with control gates for regulating the supply. A gradually inclining plane on the downstream is provided to prevent the logs from impinging on the foundation with a high velocity. The actual inclination of the platform depends upon the availability of a pool of water on the downstream. The width of the chute depends upon the number of logs that are to be passed at a time. Where timber traffic is considerable timber booms would be necessary to guide the logs head on towards the chute.

13.5. Shearing off of Pier Reinforcement

When the piers are under construction, the pier reinforcement should be properly protected during floods. Generally they are bent before the floods, and later on straighteneds when construction work is resumed. Sometimes, it has been observed that due to oblique flow or any other cause, the reinforcement bars in some of the piers get sheared off. In those cases, the reinforcement will need to be first checked up whether they are loosely sticking out or not. After ensuring its proper bonding with the lower portion, new reinforcement would be welded on to the

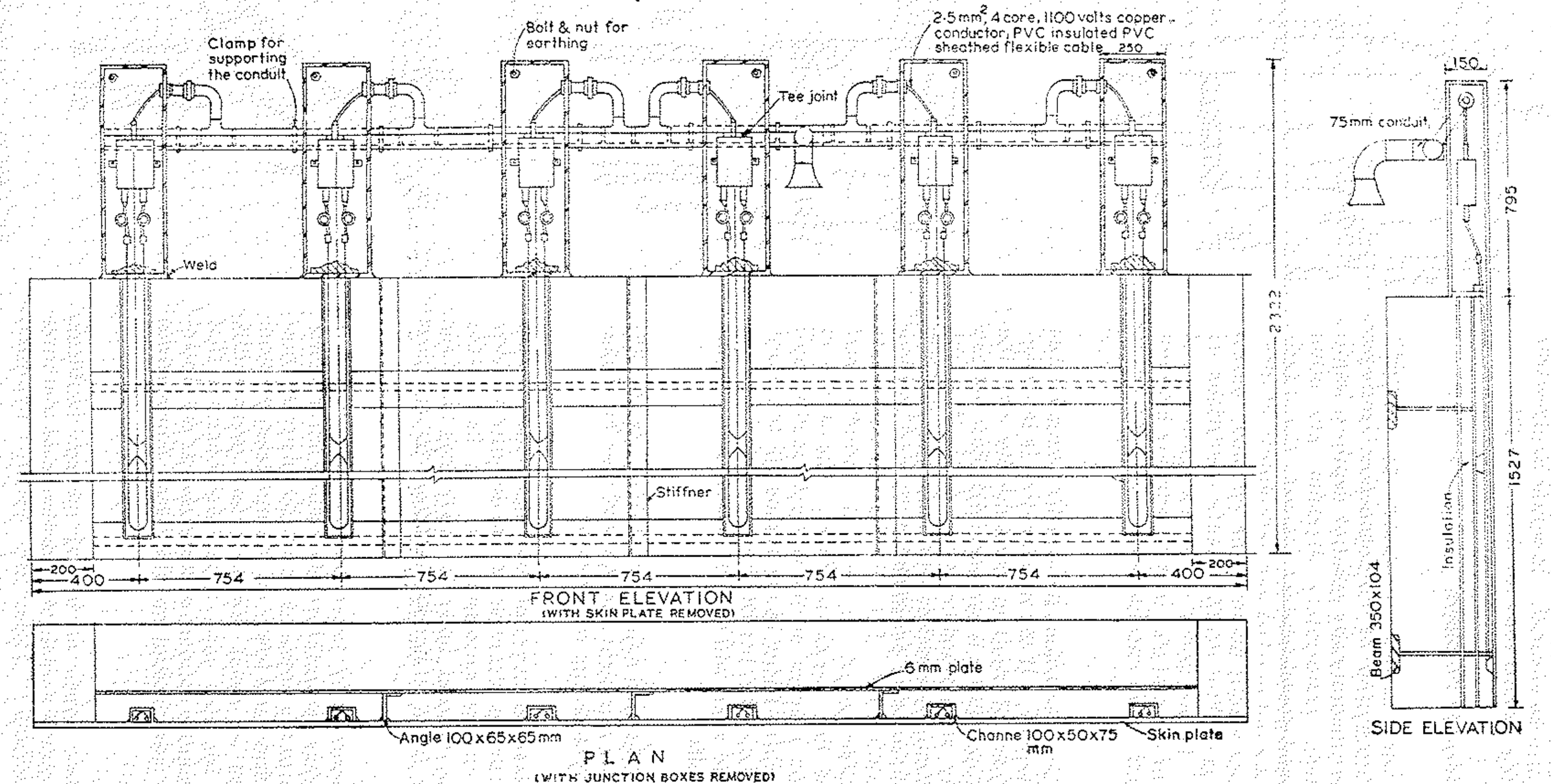


FIGURE 13.1 : Typical Details of Gate Heating Arrangement.

old ones. Proper bonding of new concrete with the old one should also be ensured. It would be advisable to test check a few welded rods for their strength.

13.6. Special Seepage Problems

In dealing with seepage pressures by Khosla's theory, the assumption of homogeneous, isotropic medium of large (theoretically infinite) depth is implicit. In practice, sedimentary deposits are often stratified. In sand gravel-boulder regions, the stratification is usually not marked and assumption of ideal Khosla conditions would normally be justified, unless rock out-crops at shallow depths create conditions of confined flow. Lower down in the trough region of the river, beds of silt and clay may be existing between layers of sand and stratification may be quite important

Exact theoretical solutions for stratified foundations become very complex. If the stratification is in numerous thin layers of random permeability and the disparity in permeabilities is not very high, flow net can be obtained by scale transformation after idealising the soil as a homogeneous but non-isotropic medium with average horizontal permeability K_H and average vertical permeability kV. This will further accentuate the relative effectiveness of vertical cut-offs and reduce that of the horizontal floor. The drope in potential will be sharper at the vertical cut-offs, and the gradient will be flatter in the region of the horizontal floor. Hence, the distribution of uplift pressures in non-isotropic soils will be more sensitive to depths of vertical cut-offs.

The effect of limited number of distinct strata has been studied by electrical analogy and also by relaxation methods by different researchers.

If the top layer is more pervious than the bottom layer, the pressures decrease on the downstream side and increase on the upstream side. In fact, in the limit, when the bottom layer is much less pervious than the top layer, this case reduces to confined flow.

When a stratum of lower permeability overlies a stratum of higher permeability, the results are reversed. Higher pressures are obtained on the downstream portion of the floor and lower on the upstream portion. For two layers of equal thickness and permeability ratio of 1/8 for this case, it has been observed that there is a maximum increase of 4 percent at the downstream end in comparison to the ideal homogenous case.

There are two types of special seepage problems usually encountered namely the confined case and three dimensional effects. These are briefly discussed below.

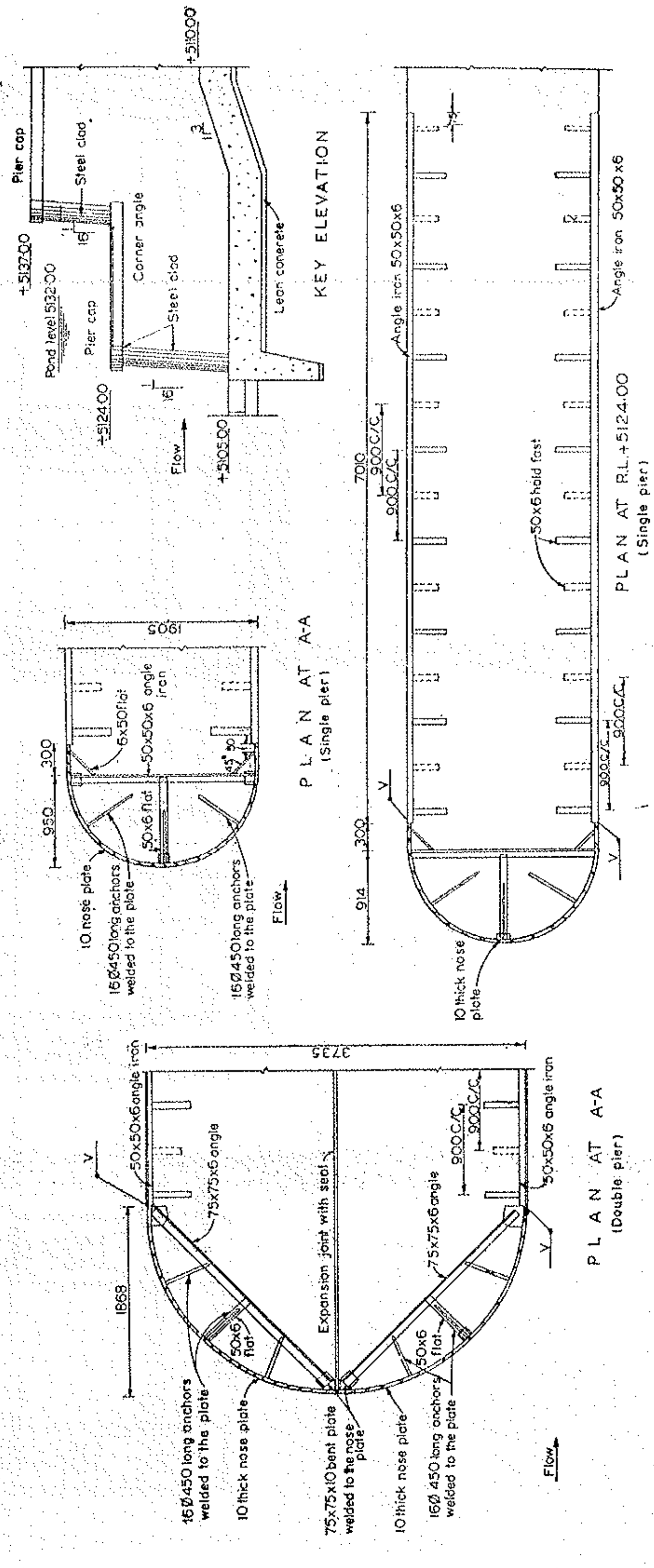


FIGURE 13.2 : Typical Steel Cladding of Pier.

13.6.1. *Confined Case* : The confined case arises when an impervious layer exists at a limited depth. Both theoritical solution and relaxation solutions have been worked out by different authors. Generally speaking confined flow results in reduction of the curvature of flow lines of seepage. The tend to become more straight and parallel. In fact if the pervious stratum becomes very narrow, the flow lines will be actually parallel to each other. Thus as depth of previous stratum is reduced, the sub-soil H.G. line approaches a straight line which means that the down-stream pressures are reduced while upstream pressures are increased.

13.6.1.1. CW&PRS, Pune, had earlier conducted some electrical analogy studies in the case of a horizontal impervious layer for finding out the change in the uplift pressure due to the different depths of pervious layer assumed in the sub-soil. The results indicated that.

(*i*) pressure at mid-point of the floor remains practically unaffected for the value of the depth of the impervious layer tested.

(*ii*) pressure at a point one-fourth the length of the floor (from upstream end) is increased to a maximum of 7 percent when depth of the layer was 0.1 to 2 percent for depth of 0.5.1 and and 0.25 percent for a depth of 1.0.1 (where 1 is the length of the floor). Similarly, at a point 3/4th of the length of the floor, the pressures were decreased to same extent, and

(*iii*) a point such as $\frac{3}{20}$th will experience a maximum increase in pressure of 9 percent due to the layer at 0.1 1 depth and the point at $\frac{17}{20}$th will undergo a similar decrease in pressure.

13.6.1.2. If the impervious layer is at a depth somewhat lower than the estimated scour depths, it may be worthwhile to tie the upstream sheet-pile line to the imprevious layer while the downstream sheet pile line may be left above the impervious layer for release of such uplift pressures as may build up by leakage through the upstream pile line.

13.6.1.3. All sheet pile lines are leaky to lesser or greater extent depending on the tightness of their joints and possibility of tearing of damage during driving. Some observations taken at actual sites indicate that the resistance to seepage offered by sheet piling has actually been equivalent to that offerred by a length of the soil at site varying between 120 to 600 m. Thus even after tying the upstream pile line to the impervious layer, seepage pressures will build up under the floor in course of time. Their magnitude is difficult to determine and a conservative design would require the assumption that a gap exists between the upstream sheet pile and the impervious layer.

13.6.1.4. If the scour situation demands such depth of downstream sheet pile line that it connects with the impervious layer, the problem becomes more difficult. It the soil under the work is boxed on all sides by the provision of cross-sheet piles also and if the sheet piles were truly water-tight and the bottom layer fully impervious, no seepage pressures should develop (This latter condition is more or less satisfied in the case of rock). Under such situations, provision of a practical floor thickness (not determined by seepage pressures) would normally be justifiable. However, pressure relief arrangement by intermediate filter or drainage gallery at one or more positions in the downstream portion of the floor would be desirable and in some cases even necessary as a precaution.

13.6.2. Three Dimensional Effects

Barrages are usually wide (across the river) in comparison to their length (along the river). For such structures, effects of three-dimensional seepage are not likely to be important.

13.6.2.1. Investigation of this problem by analytical and electrical analogy methods has indicated that the main causative factor in this problem is the elevation of the stable water table on the sides of the work. If the water table is at the same or somewhat lower level than the mean between upstream and downstream water levels, the seepage pressures are practically the same as in the two-dimensional case. If the water table is higher, the seepage pressures increase ; if it is lower they decrease. But for downstream floor, the design of which is actually controlled by uplift pressures, the possibility of the uplift pressures being less than those computed on assumption of two dimensional seepage is rather low. Near the river the water table, is at least seasonally, likely to be higher than the river bed.

13.6.2.2. The three dimensional effect will be more for works on those canals, where the water table is considerably lower than the canal bed. If so, the seepage pressures on three-dimensional basis would be appreciably lower than those on the usual two dimensional basis.