CHAPTER 5

Canal Head Works

5.1 INTRODUCTION

An irrigation main canal takes its supplies from a stream or a river. To divert the water in the canal, it is necessary to construct works across the river and at the head of the off-taking channel. These works are termed as 'Head Works'. The works constructed at the head of the canal is termed as 'Head Regulator' and its purpose is to control the supplies of water and entry of silt in the offtaking canal.

Canal head works may be divided into two classes :

(1) Diversion works
(2) Storage works

Diversion works are located across the river so as to raise the normal water level of the river to divert the required supply into the canal. These works are known as weir or barrage.

Storage works in addition to diversion, store surplus water when available in the river in excess of demand and supplement the direct flow of a river during keen demand. The present chapter deals in diversion works while storage works are dealt in section III of this book.

5.2 TYPES OF DIVERSION WORKS

The diversion works may be divided into two classes :

(1) Temporary, and
(2) Permanent weirs and barrages

Temporary spurs or bunds are constructed every year after floods. Such temporary spurs or bunds are possible with due economy in certain cases. At Katapather in district Dehradun,

Uttar Pradesh, temporary works are constructed across river yamuna to divert waters into Doon canal system.

Permanent works comprising weirs or barrages are constructed for all important head works of canal system in this country.

5.3 LOCATION OF A WEIR

Weirs are generally located in boulder or alluvial stage of the river. The location of a weir is governed by the command of the land proposed for irrigation. However if suitable sites for locating a weir is available both in boulder and alluvial stage of the river, the choice would depend upon the most economical arrangement which has to be arrived at after evaluating the advantages and disadvantages of the alternative sites.

The following are the advantages and disadvantages of locating a weir in boulder stage in comparison to alluvial stage :

Advantages

(i) The length of weir is generally shorter in boulder reaches.

(ii) Since the silt factor is high in boulder reaches, the requirements of cut-off and protection works are reduced.

(iii) Since, high banks are available in boulder reaches, the cost of training works is reduced.

(iv) Construction materials like sand and aggregate are locally available.

Disadvantages

(i) In boulder stage there may be excessive loss of water due to subsoil flow if loose boulders to great depth, are available in the river bed.

(ii) In the head reaches the canal may pass through similar terrain causing excessive absorption losses.

(iii) From take off to the ridge, the canal may cross a number of drainages.

(iv) Surface protection is necessary against erosive action of rolling boulders in floods.

(v) Demand for irrigation in the areas in boulder stage is generally low due to higher rainfall and smaller area available for irrigation.

Selection of actual site : Having decided upon the location of the weir, the actual site is selected with the following considerations :

(i) A narrow, well defined channel with banks is the best.

(ii) The canal alignment should enable suitable command without excessive digging.

(iii) Availability of materials of construction like sand and gravel.

(iv) Accessibility of the site by rail and road.

(v) Arrangement of diversion of the river during construction.

5.4 TYPES OF WEIRS

The works may be divided into the following main classes :

1. Vertical drop weirs,
2. Rock fill weirs and
3. Concrete weirs with sloping glacis.

short notes

Vertical drop weirs : All old headworks such as Bhimgoda, Madhopur, Rasual, Khanki and Marala are examples of this type of construction. A typical cross-section of a vertical drop weir is shown in fig. 5.1.

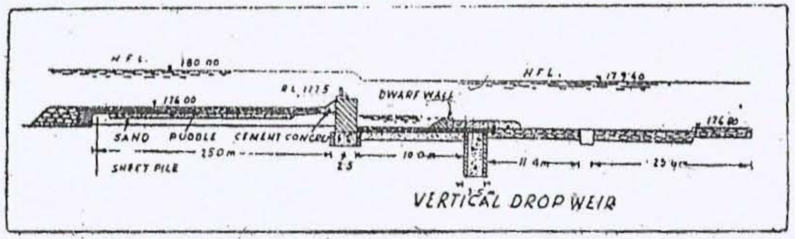

Fig. 5.1 Typical cross section of a vertical drop weir

These weirs consist of :

(a) an upstream curtain wall

(b) fore apron

(c) masonry crest with vertical or nearly vertical downstream face,

(d) downstream floor,

(e) downstream curtain, and

(f) downstream rip rap.

Upstream curtain used to be shallow about 2 m deep. The length of fore apron and downstream floor length was determined by Bligh's theory.

Part of the raising up of the water level is usually carried out by shutters at the top of the crest which are dropped down during flood. The drowned weir formula, though not very accurate, may be used for calculation of discharge.

Rock fill weirs : This class is exemplified by the one constructed at Okhla on Yamuna river near Delhi whose cross-section is shown in fig. 5.2. In this type, the boulders are laid in the form of glacis on the upstream and downstream with a few intervening walls. The downstream slope is generally made very flat. It is the simplest type of construction and many such works are of great antiquity. Their stability or principles of design are not susceptible to any theoretical treatment.

This type of weir requires a very large quantity of stone, both at the time of initial construction and during subsequent maintenanoe. Its use is, therefore, restricted to places where stone and unskilled labour is available in abundance.

Concrete weirs with sloping glacis : Weirs of this type are of recent origin and their design is based on modern concept of sub-surface flow. A typical cross-section of this category is shown in fig. 5.3. Sheet piles of sufficient depths are driven at the ends of upstream and downstream floor. Sometimes an intermediate pile line is also provided. The hydraulic jump is formed on the glacis to dissipate the energy of overflowing water. Weirs exclusively of this type are now being constructed.

IMP.

Barrages : If the difference between the pond level and crest-level is within 1.5 m, the pond level can be maintained by means of falling shutters. However if the difference is more than 1.5 m, a gate controlled weir is necessary which is called a "Barrage" With this type a roadway can be constructed

(7) River training works

Undersluices : These are gate controlled opening in the weir with crests at a low level. They are located on the same sides as off-take canal. If two canals take off on either side of the river, it would by necessary to provide undersluices on either side.

The usual functions of the undersluices are :—

(i) to preserve a clear and defined river channel approaching the canal regulator,

(ii) to scour silt deposited in front of canal regulator and control silt entry in the canal,

(iii) to facilitate working of weir crest shutters or gates. The winter floods are passed without dropping the weir shutters,

(iv) to lower the highest flood level by providing greater discharge per metre length than the weir.

Discharge capacity of undersluices is provided higher of the following :

(i) Two times the maximum discharge in the off-take canal.

(ii) 20% of maximum flood discharge.

(iii) Maximum winter discharge.

Canal head regulator : A canal head regulator serves the following functions :

(i) Regulates the supply of water in the canal.

(ii) Controls the entry of silt in the canal.

The head regulator is normally aligned between 90° to 120° in respect to the axis of the weir. The regulation is done by means of gates. The old head regulator had a large number of small span gates. The modern trend is to use steel gates of spans ranging between 8 to 12 metre and operated by electric winches.

The height of gates is determined by the difference in the crest level and the pond level. To check the flood water entering the canal, a breast wall between the pond level and H.F.L. is often provided. Provision of breast wall is usually economical than high gates unless the difference in the pond level and high flood level is nominal.

Control on the silt entering the canal is provided by keeping the crest of the head regulator about 1 m to 1.5 m higher than the crest level of the undersluices. If silt excluder is provided, it is necessary to further raise the crest of the head regulator by a minimum of 0.75 m.

Head regulators are generally provided with a very wide and shallow waterway and the drowned weir formula as given below is used to calculate the discharge.

$$Q = \tfrac{2}{3} C_1\, l \sqrt{2g}\, [(H+h_a)^{3/2} - h_a^{3/2})] + C_2 . l . d \sqrt{2g(H+h_a)}$$

C_1 and C_2 are numerical coefficients whose values may be taken as 0.577 and 0.80 respectively.

H = difference of upstream and downstream water levels.

h_a = head due to velocity of approach

l = clear length of waterway

d = depth of downstream water level above the crest.

Sometimes, the waterway at the head regulator may work out more than width of the canal. In such cases the crest level is so adjusted as to keep the waterway equal to the width of the canal. In exceptional cases waterway more than the canal width may be provided with a flared wall in the downstream of the regulator to join the canal width.

The principles of design are the same as applicable to the weir for determining the total length and thickness of floor and protection works. Usually the most critical condition of uplift pressure occurs when high flood is passing down the weir and there is no flow in the canal.

Divide wall or Groyne : It is a wall between weir and undersluices extending a little upstream of canal regulator and in the downstream upto end of loose protection of the undersluices. Normally, it is a concrete or masonry structure with top width of 1.5 to 3 m and aligned at right angles to the weir axis. The main functions of the divide wall are :

(i) to separate the floor of scouring sluices which is at lower level than the weir proper

(ii) to isolate the pockets upstream of the canal head regulator to facilitate scouring operation.

(iii) to prevent formations of cross currents to avoid their damaging effects. Additional divide walls are sometimes provided for this purpose.

Divide walls are costly structures. A large number of divide walls were provided sometimes back in barrages with a view to control cross currents. This function of the divide wall is not fully established. The modern trend is to provide divide wall to separate undersluices from weir only. These walls are likely to be subjected to maximum differential pressure when the full discharge of the river is passing through the undersluices and no discharge is passing through the weir. In this condition there will be difference in the water level on the two sides. Also there may exist difference in silt pressures on the two sides. The discharge passing down the undersluices may flush off the silt. The values of differential pressure are taken arbitrarily say 1.0 m for water heads and about 2.0 m for silt pressure.

Short note

These walls are founded on wells closely spaced beyond the pucca floor upto the end. Typical cross section of the divide wall on pucca floor and heads are given in fig. 5.5 (i) and 5.5 (ii).

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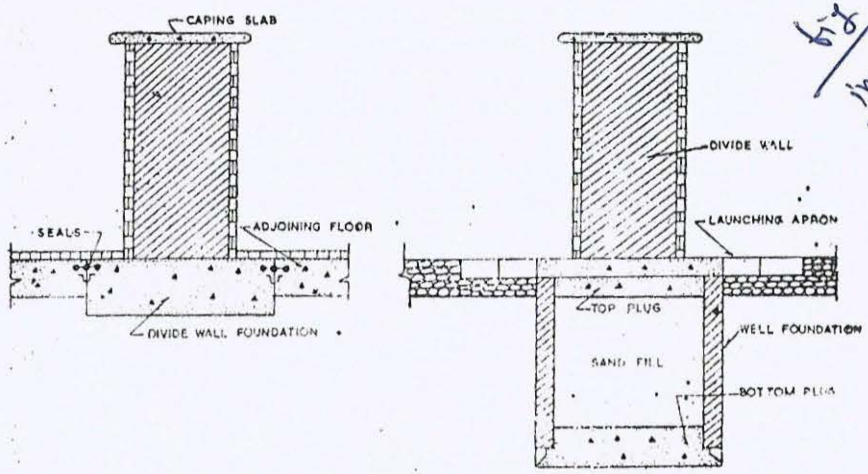

(ii)

Fig. 5.5 (i) Cross section of divide wall on pucca floor.
(ii) Cross section of divide wall beyond pucca floor.

Fish Ladder : Fish ladder or fish passes are generally provided to enable the fish to ascend the head waters of the river and thus reach their spawning grounds for propogation or to follow their migratory habits in search of food.

The general requirements of a fish ladder are :

(i) The slope of the fish ladder should not be steeper than 1:10, so as to ensure a current of velocity not exceeding 2 m per second in any portion of the fishway,

(ii) The compartments of bays of the pass must be of such dimensions that the fish do not risk collision with the sides and upper end of each bay when ascending.

(iii) Plenty of light should be admitted in the fishway.

(iv) The water supply should be ample at all times.

(v) The top and sides of a fishway should be above ordinary high water level.

Fish ladders are generally located adjacent to divide wall near undersluices because there is always some water in the river section below them.

The various types of fish ladders are : (i) pool type (ii) steep channel type (iii) fish lock and (iv) fish lift or elevators type. Types (iii) and (iv) are suitable for high dams only. Types (i) and (ii) are generally provided in barrages. The main problem in the design of fish passes is to dissipate energy in

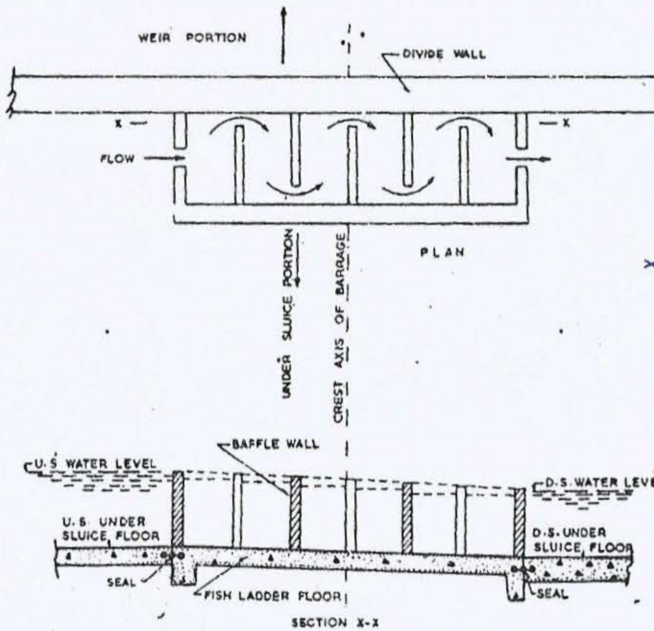

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Fig. 5.6 Typical cross-section of a fishway such a manner as to provide smooth flow at sufficiently low velocity. A typical fish way is shown in fig. 5.6.

Piers and Abutments

In barrages piers are provided at an interval of 10 to 20 m. The piers support bridge decking, and working platform for the operation of gates. Usually the cutwaters are simple in shape. The side faces of the piers are often vertical. Tapering if done, does not exceed 1/50 to 1/40. The piers are constructed in concrete or masonry and are slightly reinforced to resist the forces of water to which they are subjected. Piers should be provided with separate foundations. A continuous rubber or copper seal is provided between pier and gravity floor to ensure water tightness. A typical cross-section showing pier foundation and sealing arrangement is shown in fig. 5.5 (i) In case, however, when a raft is provided, the piers may be constructed monolith with the floor.

Abutments are usually gravity section and founded on wells packed closely in either direction. Perhaps in case of higher abutments a better alternative is to provide a counter-fort type structure on open foundation. At barrage I of Sharda Sahayak Pariyojna in U.P. settlement of about 10 mm for abutment was anticipated and counterfort retaining structure was preferred to a gravity abutment founded on wells. To safeguard against undue differential settlement portions of the abutment with differential loadings. have been separated and sectionalised by 13 m deep sheet piles.

Protection Works

The pucca floor of a weir or barrage is protected on the upstream as well as downstream by loose apron. In the immediate vicinity of pucca floor a certain portion of the loose apron is made non launching. The non launching apron prevents the scour hole to travel close to the floor launch or sheet pile line ; whereas launching apron is designed to launch along the slope of the scour hole to prevent further scooping out of the underlying river bed material.

The detailed design of the apron has been discussed in Article 2·14 and also in the chapter on river training works.

River Training Works : Generally the following river training works are provided on canal head works.

short note

Fig imp

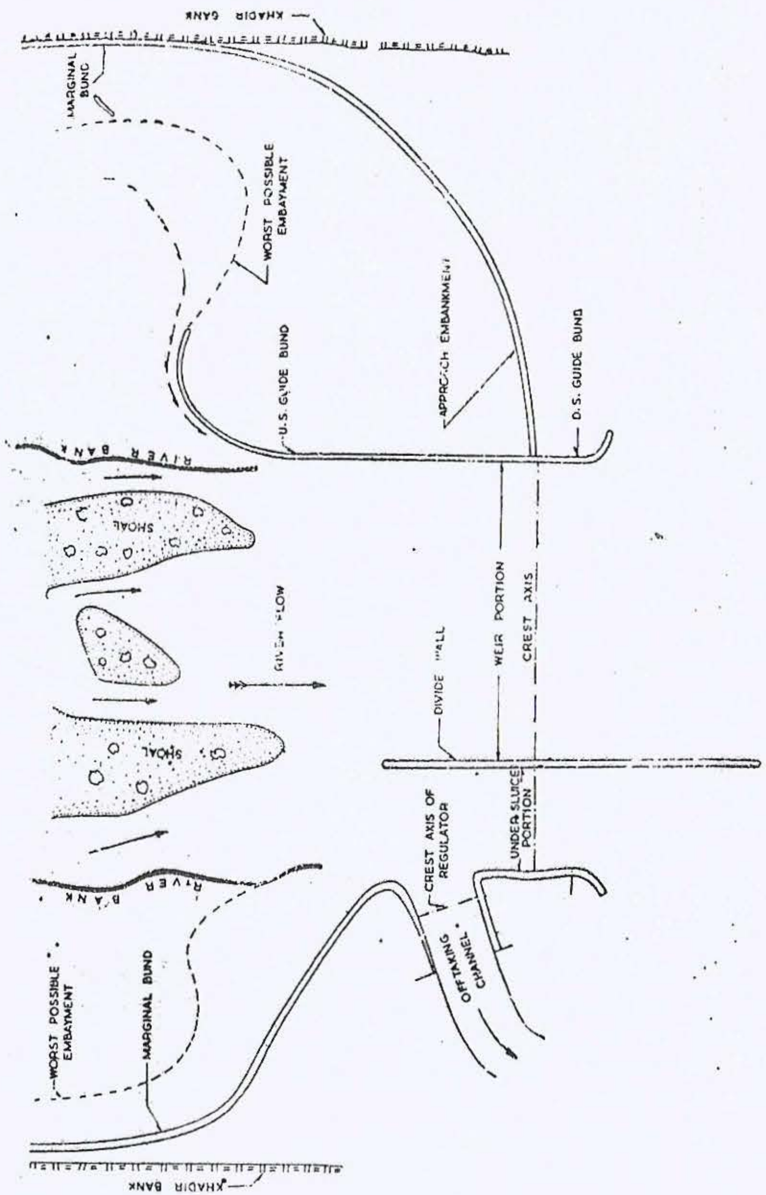


Fig. 5.7 Layout of barrage in relation to approach embankment, off take channel and worst embayment

(i) Marginal or afflux bunds
(ii) Guide bunds, and
(iii) Spurs and groynes.

Marginal bunds are provided to protect the land and property against submergence during ponding or high stage of the river. Construction of marginal bunds would be justified only when the value of land saved is more than the cost of the marginal bunds. In some cases when the ponding is high and the water shed is low, the construction of marginal bunds would be obligatory.

The layout of marginal bunds is very important in economising the overall cost of the training works and subsequent cost of maintenance. The length of the guide bunds depends very much on the layout of the approach embankment and marginal bunds. The general tendency is to run the afflux bunds parallel and near the river bank. Such afflux bunds come within the zone of meander and are frequently attacked by the river and have to be protected at heavy cost by providing spurs. It is, therefore, recommended that efforts should be made to design the approach embankment on the side and the upstream bank of the off-taking channel itself as an afflux bund upto the khadir edge or a reasonable distance so as to accommodate the worst embayment as shown in fig. 5.7. The design of marginal bunds depend upon its height and soil characteristics. Generally homogeneous embankment section is adapted with 2 : 1 slope both upstream and downstream. However, high embankments should be designed on the principles of earth dam. The design of guide bunds and spurs is dealt in chapter 18 of Vol I.

Short note

5.6 EFFECT OF CONSTRUCTION OF A WEIR ON THE RIVER REGIME

The sediment transporting capacity of water depends mainly on the discharge, slope and grade of material. Due to creation of an obstacle in the river bed in the form of a weir, the river regime will be affected in the following sequence (Fig. 5.8).

(i) A weir will pond up water leading to the flattening of water surface slope for some distance on the upstream side.

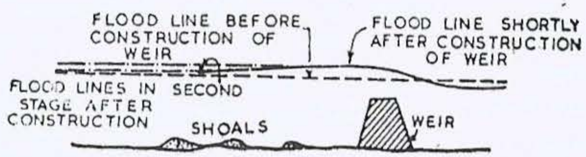


Fig. 5.8 Effect of construction of a weir on river regime

(ii) As a result of flattening of water slope, the river would drop a part of its sediment load resulting in the formations of shoals in the pond.

(iii) While sediment is dropped in the pond, relatively clear water passes over the weir. This water scours the river bed to make up the deficiency in its silt load and causes a progressive lowering or retrogression of downstream levels.

(iv) The progressive silting and formation of shoals in the upstream increases the resistance to flow of water. To overcome this resistance increased head is required and the river starts to regain its original slope. A stage is gradually reached when the pond absorbs no more silt.

(v) As a consequence, the normal sediment load is passed down the weir. In addition, since the off-take canal takes comparatively sediment free water, the sediment load remaining the same, the discharge decreases. This will lead to sediment deposit in the downstream. After a few years, it not only regains its original level but starts silting up.

(vi) The overall effect of a weir is to take away the excess energy due to steep gradient by localising it and ultimately dissipating it.

The effect of retrogression should be given due consideration in the design as explained later.

5.7 FAILURE OF WEIRS ON PERMEABLE FOUNDATION

The causes of failures may be classified into two broad categories :

1. Due to seepage or subsurface flow

2. Due to surface flow.

The subsurface flow endangers the stability of a weir in the following two ways.

Piping or floatation: If the seepage water has sufficient residual force at the end of the work to lift up soil particles, it will lead to progressive removal of soil beneath the foundation of the work resulting in its failure.

Uplift pressure: The seepage water exerts an upward or uplift pressure on the floor of the works. If the thickness of floor is not adequate to withstand the uplift pressure, it would result in the failure of the floor.

The surface flow also endangers the weir in the following two ways:

Unbalanced head due to standing wave: Due to the formation of the jump very high unbalanced pressures are developed in the trough. The floor should, therefore, be of adequate thickness to withstand the effect of the jump.

Scour on the upstream and downstream: In floods the beds of alluvial rivers are scoured to considerable depths. If the scour on the upstream and downstream is allowed unchecked, it may cause considerable damage to the works due to undermining.

5.8 CONSIDERATIONS IN THE DESIGN OF WEIR OR BARRAGE

Technique of weir design has been evolved largely on the study of causes which led to their failures.

The design of weir or barrage like any other hydraulic structure consist of the many phases as already dealt in article 1.13 and are summarised in Table 5.1. The two main phases are:

1. The hydraulic design.
2. The structural design.

The former deals with the evaluation of the hydraulic forces acting on the structure and determination of the configuration of the same for the best economy and functional efficiency. The structural design consists of dimensioning of the various parts of the structure to enable it to resist safely all the forces

Table 5.1

Design of Weir or Barrage →

- Hydraulic design →
  - In respect to sub soil flow →
    - Determination of uplift pressures
    - Determination of exit gradient
  - In respect to surface flow →
    - Crest levels of weir and under sluices
    - Determination of optimum waterway, afflux and discharge
    - Evaluation of effect of retrogression
    - Design on the consideration of hydraulic jump →
    - Design with consideration of scour →
- Structural design
  - Gravity floor
  - Raft design

→ Determination of downstream floor level

Determination of length of downstream floor

→ Determination of uplift pressures

→ Design of protection works.

Determination of upstream and downstream sheet piles.

→ Determination of minimum depth of upstream and downstream acting on it. This is done by the accepted norms of structural analysis. The problem involved in the hydraulic design of weirs on permeable foundation may be treated under the following categories.

1. Sub soil flow and
2. The surface flow.

The design aspect of hydraulic structures on permeable foundation in respect to subsoil flow has been treated in detail in chapter I and summarized later in article 5.11. The design aspect with consideration to surface flow is dealt here in subsequent para 5.9.

5.9 DESIGN WITH CONSIDERATION OF SURFACE FLOW

Crest level : The crest level of a barrage is fixed on the consideration of the existing river bed levels at the proposed site. The undersluice crest is usually kept as near the bed level in the deepest channel as is practically possible. The barrage bay crest is kept slightly higher and at about the general bed level in the remaining portion of the river. The undersluice crest is kept lower to attract a deep current in front of the regulators so that the dry weather current may remain near the regulator.

It shall be seen that the afflux and discharge per metre are related to the crest levels. Lower crest levels result in lesser afflux but higher discharge per metre. A low set barrage with increased depth of water over the crest may result in an increased height of gates, thickness of floor, and cost of superstructure above floor level.

Having tentatively decided the crest levels as well as the waterway and afflux of the undersluices and weir proper, it is necessary to check that the maximum flood discharge passes down the works without exceeding the afflux decided. The following discharge formulae may be used for this purpose.

(a) For broad crested weirs :

$$Q = 1.705\ (L - 0.1\ nH)\ H^{3/2}$$

(b) For sharp crested weirs :

$$Q = 1.84\ (L - 0.1\ nH)\ H^{3/2}$$

where

L is total clear waterway in m.

n is the number of end contractions.

H is the head over the crest in m.

Due to improvement of coefficient of discharge a raised crest should be provided whenever possible. In case of submargence the value of coefficient be reduced as per figure 6.5.

Afflux, Length of Waterway and Discharge Intensity

As a result of putting obstruction across a river in the form of a weir, the maximum flood level of the river upstream of the weir rises. This rise of level is termed 'afflux'. Afflux actually denotes loss of head and its magntiude is represented by the difference in total energy levels on the upstream and the downstream of the works.

The length of waterway, corresponding discharge per metre and afflux are co-related. By providing higher afflux, the length of the weir can be reduced but the cost of weir and training works will increase due to increased head of water. These parameters are decided after consideration of many practical aspects such as effect of back water on existing structures and submergence of land. It shall be seen that cost of works as a whole is minimum for a certain waterway and afflux. Endeavour should therefore be made to attain most economical combination. This can only be done by trial and error. Afflux is generally limited to one metre but may be kept higher if permissible.

A likely figure to adopt for waterway is given by the following formula representing Lacey's wetted perimeter,

$$P = 4.83\ \sqrt{Q}$$

In boulder reaches of the river, it would be economical to reduce the waterway to about 0.6 to 0.8 times Lacey's waterway, the minimum being the actual width available between banks. In plains where the silt factor is in the neighbourhood of unity it is generally economical to keep the waterway 1.0 to 1.2 times the Lacey's waterway. It is desirable to prefer shorter waterway, which watery reduces shoaling both on the upstream and downstream of the weir. This helps in regulation of the weir.

Table 5.2 Data of barrages constructed during last twenty years

Sl. No.	Name of weir or barrage	Dak-pather	Wazira-bad	Yamuna below 'C' P.H.	Kemri	Phika	Sharda	Narora	Durga-pur	Kosi	Sone	Trishuli	Gandak
1	2	3	4	5	6	7	8	9	10	11	12	13	14
1.	Name of River	Yamuna	Yamuna	Yamuna	Pilakhat	Phika	Sharda	Ganga	Damodar	Kosi	Sone	Trishuli	Gandak
2.	State	U.P.	Delhi	Delhi	U.P.	U.P.	U.P.	U.P.	Bengal	Bihar	Bihar	Nepal	Nepal
3.	(a) D.S. H.F.L.	455.22	208.10	206.20	184.80	259.74	224.33	179.83	64.47	75.44	109.61	208.68	112.38
	(b) Afflux	1.01	0.12	0.076	0.244	1.78	1.371	0.914	0.914	2.286	1.213	0.975	0.67
4.	Catchment area in sq. km.	7340	—	—	1230.0	127.0	—	32893.0	—	61722.0	68915.0	—	37855.0
5.	Design maximum flood disoharge in cumec	14560	7079.0	8495	1416.0	1048.0	16990.0	14160.0	15574.0	26901.0	40493.0	4248.0	24069.0
6.	Cost in Iakhs of Rupees	—	—	—	40	—	—	441	455	—	995	—	—
7.	Cost per cumec in Rupees	—	—	—	2830	—	—	3100.0	2920.0	—	2458.0	—	—
8.	Lacey's water-way Pw (metre)	582.17	406.22	445.00	181.96	156.66	627.88	575 76	603.50	792.48	973.83	313.94	749.00
9.	Width between abutments (B)	517	454.09	531.6	162.8	108.0	667.0	922.5	692.20	1149.14	1412.16	142.45	748.2
10.	Ratio. B/P_w.	0.89	1.12	1.19	0.894	0.69	1.06	1.6	1.45	1.45	1.445	0.453	0.98
11.	Clear Water-way in metre												
	(i) Weir	347.80	300.53	402.31	100.60	64.0	518.16	758.4	439.0	841.20	1042.4	100.6	329 2
	(ii) Under sluices	109.73	100.6	73.15	36.58	27.4	60.96	106.7	182.9	E 109.73 W 73.15	E 73.15 W 146.30	18.23	330.7
	(iii) Total	457.53	401.13	475.46	137.18	91.4	579.12	865.1	6.219	1024.08	1261.85	118.83	659.9 of all sluice
12.	Maxm. Discharge capacity in cumec (i) Weir	10359	3540	6950	980	691	14725	11882	10307	20162	32564	3700	19737
	(ii) Under sluices	4189	2169	1557	436	357	2265	2676	R 2525 L 2690	6739	8891	543	4446 each sluice
	(iii) Total	14548	5709	8497	1416	1048	16990	14558	15522	26901	41455	4243	24070
13.	Discharge per m. run. (Average) in cumec.	28.19	12.6	16.0	8.7	9.7	25.4	15.75	22.5	23.4	29.4	29.8	32.0
	(a) Between Piers (i) Weir	29.80	11.8	17.3	9.8	10.81	28.4	15.7	23.4	24.4	31.2	37.0	32.4
	(ii) Under sluices	38.2	21.6	21.3	11.93	12.98	33.2	25.1	37.7/29.4	36.9	40.6	29.8	40.5
	(b) Out side piers (i) Weir	26.3	14.9	15.5	8.24	9.22	24.6	15.0	21.1	24.8	28.1	32.6	29.0
	(ii) Under sluices	33 8	18.5	19.6	10.81	11.45	33.0	21.4	25.3/27.4	33.0	37.0	23.4	36.1

1	2	3	4	5	6	7	8	9	10	11	12	13	14
14.	Crest Level (i) Weir	450.65	202.69	199.95	182.02	258.47	219.45	176.15	59.74	71.63	103.93	202.08	105.76
	(ii) Under Sluices	459.43	201.37	199.03	181.41	257.86	218.54	174.59	L 59.13 R 5.83	70.14	103.32	201.17	104.22
15.	Silt Factor	4.0	1.0	1.0	1.0	2.0	3.0	0.896	1.0	1.30	1.4	15.0	3.0
16.	Depth of Scour. (R) Below H.F.L. in metre (i) Weir	6.096	8.10	8.34	5.52	4.68	7.86	8.6	10.10	10.0	11.0	5.4	8.7
	(ii) Under Sluices	7.01	9.36	9.7	6.5	5.38	9.39	10.87	11.25/12.13	12.56	13.26	4.49	10.11
17.	Exit Gradient	1/5	1/8.25U.S. 1/7 Weir	1/11Weir 1/14 U.S.	1/5	1/4	1/6.15 1/6.85	1/6	1/5.3	1/5	1/5.1 1/5.77	1/6.5 1/4.03	1/4.5
18.	Weir Floor Level upstream	449.3 S1:200 U.S. crest	201.78	200.0	181.50	257.55	219.15	175.57	57.91	70.1	103.32	201.17	103.63
	D_1	7.14	6.44	6.32	3.53	3.962	6.55	5.19	7.07	7.62	7.49	8.53	9.44
	downstream	446.07	202.00	200.1	178.8	255.0	218.0	173.94	86.08	67.66	100.42	199.00	101.1
	D_2	9.33	6.31	6.30	6.20	4.0	6.4	6.1	7.99	7.77	9.18	9.51	11.2
19.	Under sluices floor level u.s.	449.43	202.00	199.0	181.5	357.5	218.5	174.5	L 57.30 R 57.00	70.1	103.30	201.0	104.1
	D_1	6.98	6.75	7.24	3.54	3.96	7.16	6.15	7.68/7.97	7.62	7.47	8.5	8.81
	d.s.	456.28	201.80	199.0	178.5	255.0	217.5	172.8	S 55.1 R 54·1	66.4	100.10	197.50	99.6
	D_2	10.54	6.62	7.16	6.49	4.31	7.01	7.31	8.9/9.2	9.0	9.33	11.2	12.7
20.	Length of floor												
	(a) Weir (i) u.s.	38.10	11.59	4.87	7.75	9.45	14.79	21.7	8.85	14.91	10.05	12.2	13.7
	(ii) d.s.	44.20	11.90	23.10	20.60	21.0	27.85	27.48	39.00	33.95	39.2	31.1	43.40
	(b) Under sluices (i) u.s.	82.3	9.15	4.87	12.34	9.45	22.10	23.39	8.85/8.85	14.95	10.05	—	U/S13.70 K/S13.70
	(ii) d.s.	48.8	21.30	23.80	20.60	21.0	44.60	34.5	43.75/44.5	39.95	39.20	—	U/S43.5 K/S46.20
	Maximum head	11.43	5.49	3.20	5.79	7.0 for R.L. 262.10	—	6.4	9.6	11.27	7.92	—	U/S10.66 R/S11.1

Data regarding crest levels, waterway afflux and other useful parameters of some of the barrages constructed in this country during the last 20 years are given in Table 5.2. This shall be usuful while designing new works.

In calculating the cistern levels and depth of sheet piles, the possibility of non-unform flow is taken into account by providing a suitable concentration factor. This factor is chosen arbitrarily. Usually 20% concentration is taken at any particular section. Therefore, active discharge per metre is 5/4 of normal discharge intensity obtained by dividing the maximum discharge by the length of crest. No concentration of flow is usually taken for designing protection works.

Retrogression

The effect of construction of a weir on the river regime is explained in Article 5.6. If retrogression or lowering of bed levels in the early stages after construction is not taken into account, it can lead to undermining of the floor. Observations made in case of many Punjab weirs have revealed a retrogression of 1.2 to 2.2 m. The retrogression of this magnitude has been observed at low water levels but at high flood levels the maximum retrogression observed is between 0.3 m to 0.5 m.

The actual design practice is to allow a retrogression of 0.5 m increasing linearly upto 2.0 m at low discharges.

Short

5.10 DESIGN WITH CONSIDERATION OF HYDRAULIC JUMP

Donot write reading point of view prepare

On a level floor with low friction, the position of hydraulic jump is unstable. For a slight change in the depth or velocity, the position of hydraulic jump will vary widely. On the other hand, the position of hydraulic jump on the sloping glacis is more stable and can be predicted closely. It is, therefore, considered essential to provide a sloping glacis so as to ensure that the hydraulic jump is confined to the sloping glacis and under no circumstances it is formed lower than the toe of the glacis.

Slopes ranging from 1 : 3 to 1 : 5 are found to be most suitable for the glacis.

Downstream floor levels as determined by the hydraulic jump : In chapter 2 the method of locating the

point of hydraulic jump was explained. The level of the downstream floor for the condition that the point of jump is not lower than the toe of glacis

$$= \text{Downstream total energy level} - E_{f2}$$

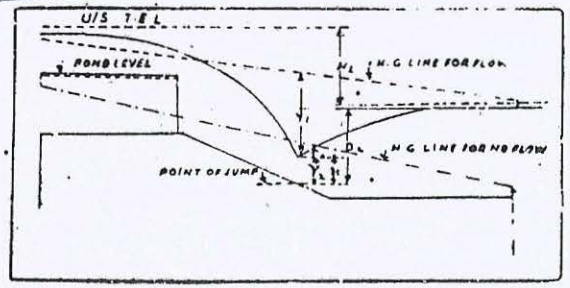
The downstream level so obtained should be tested for other conditions of flow when the discharge is lower but the retrogression may be much higher.

Length of horizontal floor as determined by the hydraulic jump : The main disturbance as a result of jump extends upto a distance of 5 (D_2-D_1) from the point of formation. In order that the filter area and stone protection be safe from the main turbulence of the jump, the minimum length of the horizontal floor should be 5 (D_2-D_1). Sometimes even 6 (D_2-D_1) is recommended for poor soils.

Design of downstream protection works-stilling basin. The value of incoming Froude number for barrages and canal regulators usually lies in between 2.5 to 4.5, zone of weak hydraulic jump. The energy dissipators and stilling basin for such conditions need careful design. Different states and organisations have recommended standard design criteria. The more common are U.S.B.R. stilling basins, R. S. Varshney's design charts for stilling basins for low Froude numbers, Saint Anthony Falls Lab. design etc. These have been discussed in detail in Chapter 2 on hydraulic jump and energy dissipation devices.

Short note as per wisdom

Uplift pressures in the region of hydraulic jump : For determining the uplift pressures in the jump trough, the complete water surface profile of the standing wave and the subsoil pressure gradient line for particular flow conditions are plotted. The uplift pressure is the ordinate measured from the hydraulic gradient line to water surface. It may be seen in fig. 5.9 that ordinate y_b, is the net uplift pressure at the location of the hydraulic jump. On the other hand y_2 is the net uplift pressure at this location which will occur with the maximum pond level upstream and no flow downstream. The requirement of the floor thickness is worked out by taking the larger of these two values and dividing it by the submerged density of the floor material, (G−1) where G is the specific gravity of the floor material. The floor at every section



jump

Fig. 5.9 Uplift pressures on floor—Definition sketch

has to be designed for the larger of the two uplift pressures (i.e. one with hydraulic jump and the other for no flow condition). The uplift pressures due to hydraulic jump are further reduced because of the following reasons :

(i) The backward rolling flow of water in the trough reduces the uplift pressure.

(ii) The uplift pressure due to standing wave is maximum at the point of jump formation but reduces rapidly on either side. As the floor has beam action, it may be designed for average uplift pressures than the maximum occuring at a point.

(iii) The vertical component of the momentum remains unaffected in the jump which exerts a downward pressure in the vertical direction. The magnitude of this force is quite considerable.

Due to the above mentioned factors, the uplift pressure for design purposes due to jump formation are taken 2/3 of the theoretical value. Uplift pressures at the point of jump formation may also be taken as 50% $(D_2-D_1) + \phi H_L$ where ϕ is the percentage of pressure at the jump location.

The design with consideration to scour has already been dealt in para 1.15 of Chapter 1 on design of structures on permeable foundations.

5.11 THE DESIGN OF BARRAGE (Stepwise procedure) Fully (All steps)

imp

viva

The following data must be known

(i) Maximum flood discharge (Q)

(ii) Stage discharge curve of the river at barrage

(iii) Minimum water level

(iv) Cross section of the river at barrage site

The following have to be decided

(i) Lacey silt factor (f). This is determined from the equation, $f = 1.76\sqrt{M_r}$

(ii) Length of waterway, discharge per metre and afflux

(iii) Safe exit gradient

(iv) Depth of sheet piles in relation to (i) scour depth and (ii) exit gradient.

(v) Level and length of horizontal part of downstream impervious floor in coordination with hydraulic jump.

(vi) Thickness of downstream impervious floor :

(a) with reference to uplift pressure,

(b) with reference to hydraulic jump or standing wave.

(vii) Length and thickness of protection works beyond pucca floor upstream and downstream.

Procedure

STEP I Determine head loss (H_L) for different flow conditions :

If there is no retrogression H_L = afflux

If allowance for retrogression is taken in downstream bed level, then H_L = afflux + retrogression. Usually 0.5 m retrogression will be sufficient in most cases.

STEP II For known values of q and H_L read corresponding values of E_{f2} from Blench curves (Fig. 2.5). With known values of E_{f2} read corresponding values of D_2.

Cistern level = Downstream T.E.L. $- E_{f2}$

STEP III $E_{f1} = E_{f2} + H_L$. Knowing E_{f1}, E_{f2}, and q, read values of D_1 and D_2 from Fig. 2.7, energy of flow curves. Provide minimum cistern length $= 5\,(D_2-D_1)$.

STEP IV Determine scour depth from the formula

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

Depth of upstream sheet pile from scour consideration

$= 1\,R$ to $1.25\,R$

Depth of downstream sheet pile from scour consideration

$$= 1.25 \text{ R to } 1.5 \text{ R}$$

An intermediate pile line need not normally be provided. If at all provided, its depth should not be less than that of the upstream pile line.

STEP V Work out the value of $\frac{1}{\pi\sqrt{\lambda}}$ from the equation $\frac{1}{\pi\sqrt{\lambda}} = G_E \frac{d}{H}$ for the given value of G_E and the known values of d (downstream depth of sheet pile) and H (Maximum static head). Corresponding the value of $\frac{1}{\pi\sqrt{\lambda}}$ read the value of α from figure 1.12.

STEP VI Provide total length of floor (b) $= \alpha$ d. Disposition of total floor length may be as follows:

(1) Cistern length $= 5\ (D_2 - D_1)$ to $6\ (D_2 - D_1)$

(2) Glacis length $=$ 3 to 5 times (crest level $-$ cistern level). for 3 : 1 to 5 : 1 slope of glacis.

(3) upstream floor $=$ the balance.

If the total length is excessive, it would be economical to reduce it by providing a deeper downstream sheet pile.

STEP VII In order to determine uplift pressures acting on the floor, the % pressures at upstream and downstream sheet pile lines are worked out. The pressure distribution from upstream sheet pile line to downstream sheet pile line is assumed to be linear.

% pressures at upstream sheet pile line.

For the known values of b and d_1, $\frac{1}{\alpha} = \frac{d_1}{b}$. Having known α read out values of ϕ_D and ϕ_E from plate no. 1.

% pressure at the bottom of sheet pile $= 100 - \phi_D$

% pressure at the bottom of floor $= 100 - \phi_E$

% pressures at downstream sheet pile line.

From the known values of b and d_1. $\frac{1}{\alpha} = \frac{d_1}{b}$, read values of ϕ_E and ϕ_D corresponding to $\frac{1}{\alpha}$ from plate no. 1, which

would be % pressure at the bottom of floor and sheet pile respectively.

% pressures at intermediate pile line.

For the known values of the depth of intermediate pile line (d_3) and total floor length b, determine $\alpha = \frac{b}{d_3}$. Also calculate the base ratio

$$\frac{b_1}{b} = \frac{\text{Horizontal distance between upstream pile and intermediate pile}}{\text{Total floor length}}$$

The avlue of ϕ_C can be read directly from plate no. 1 for given values of α and base ratio $\frac{b_1}{b}$. To find ϕ_E for the known value of α and base ratio $\frac{b_1}{b}$, read for base ratio $1 - \frac{b_1}{b}$ for that value of α, and subtract from 100. To get ϕ_D for $\frac{b_1}{b}$ less than 0.5, read ϕ_D for base ratio $1 - \frac{b_1}{b}$ and subtract from 100.

Correction due to floor thickness

The thicknesses of the floor at the location of the sheet piles are tentatively assumed for correcting the values of ϕ_C in the upstream and ϕ_E in the downstream. If t_1 is the floor thickness at upstream sheet pile of depth d_1, correction due to floor thickness $= \frac{t_1}{d_1}(\phi_D - \phi_C)$ which is positive. If t_2 is the floor thickness at downstream sheet pile of depth d_2, the correction $= \frac{t_2}{d_2}(\phi_E - \phi_D)$ which is negative.

Correction due to mutual interference of sheet piles

The correction due to mutual interference of sheet piles is worked out by the following formula:

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

Slope correction

This is applicable only in case where an intermediate pile line is provided. The values of correction are given in chapter 1.

STEP VIII After knowing the corrected percentage pressure under the key points, the sub-soil pressure gradient line and hydraulic gradient line for surface flow is plotted with reference to the corresponding downstream water level as datum. The corresponding water profiles before and after the jump formation are also plotted for the given value of discharge intensity, q.

Knowing q and E_{f1} at different locations of the glacis, corresponding values of D_1 are read from figure 2.7 and thus the water profile before jumb formation can be plotted. For plotting water profile after jump, the Froude number (F) is determined from the formula $= F = \frac{q}{\sqrt{gD_1^3}}$. Knowing F^2, the relation between the abscissa and ordinate of the profile can be read from the figure 2.8.

The intercept between the profile of hydraulic jump and the gradient gives the unbalanced head. The floor thickness is, however, designed for 2/3 the maximum unbalanced head in the jump trough.

The uplift pressure which will occur with the maximum pond level upstream and no flow downstream should also be determined. The requirement of floor thickness is worked out by taking the larger of the two uplift pressures and diving it by the submerged density of the floor materia (G—1). Since the floor is generally of cement concrete G is taken as 2.24 for safer design.

STEP IX The protection works are now designed in respect to the scour depth as per recommendations given in Article 1.15.

The detailed design of the weir, undersluices and head regulator are illustrated in the subsequent articles.

5.12 EXAMPLE ON DESIGN OF BARRAGE

A barrage is to be constructed on a river having a high flood discharge of approximately 10,000 cumec.

12,000

The relevant data are as follows :

Average bed level of river	~~299.50 m~~ 300
High flood level (before construction of barrage)	~~305.00 m~~ 306
Permissible afflux	1.00 m 1.0
Pond level	~~303.00 m~~ 304.
Lacey's silt factor	1.0 1.0
Safe exit gradient for river bed material	~~1/6~~ 1/7
Concentration	20% 20%
Bed retrogression	0.50 m 0.5 m

Stage discharge curve of the river at barrage site is given in fig. 5.10.

Prepare a complete design for the headworks on the basis of the formation of hydraulic jump and uplift pressures etc. by Khosla's theory.

I. FIXATION OF CREST LEVELS AND WATERWAYS

The upstream floor level of the undersluices is generally kept at the average bed level of the river.

Keep the upstream floor and the crest level of undersluices = 299.50 m. The crest level of the other barrage bays is kept 1.0 m to 1.50 m higher than the crest level of the undersluices.

Adopt crest level of the other barrage bays 1.1 m higher than that of the undersluices.

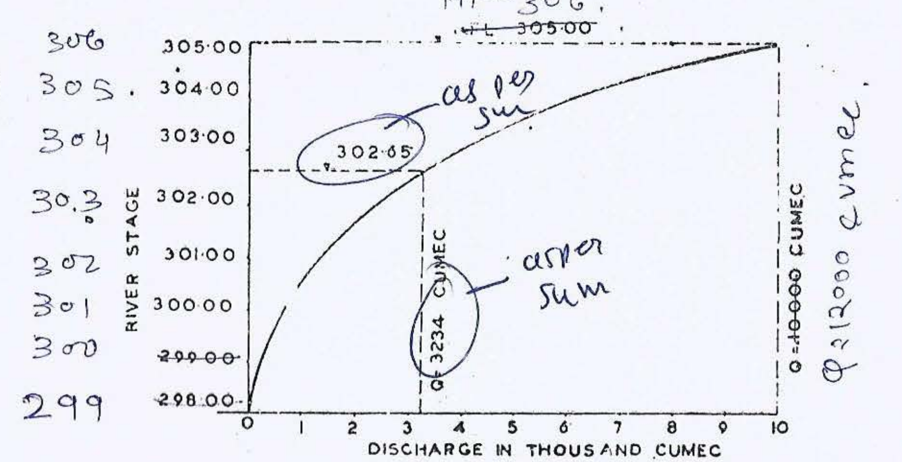


Fig. 5.10 Stage discharge curve

Thus crest level of other barrage bays = 300.60 m

As per Lacey's formula, minimum stable water way

$P_w = 4.83\ \sqrt{Q} = 4.83\ \sqrt{10{,}000}$

$= 483$ m

The actual waterway of the barrage is decided by trial and error on the basis that about 20% of the maximum flood discharge should be able to pass through the undersluices, and the balance through other barrage bays.

Assume the waterway as follows :

(a) *Undersluices portion*

5 bays of 15 m each	75.0 m
4 piers of 2.5 m each	10.0 m
Overall waterway	85.0 m

(b) *Other barrage bays portion*

30 bays of 12.0 m each	360.0 m
29 piers of 2.0 m each	58.0 m
Overall waterway of other barrage bays	418.0 m
Assume one divide wall thickness	3.0 m

Thus assumed overall waterway of the barrage between abutments = (85+418+3) = 506.0 m.

Now it shall be checked whether the maximum flood can pass through the assumed waterway.

H.F.L. before construction of barrage	305.0 m
Permissible afflux	1.0 m
Thus u.s. H.F.L. (d.s. H.F.L. + Afflux)	306.0 m

Average discharge intensity $= \dfrac{10{,}000}{506} = 19.76 \approx 19.75$ bumec

Scour depth 'R' $= 1.35\ (q^2/f)^{1/3} = 9.85$ m

Velocity of approach $= q/R = \dfrac{19.75}{9.85} = 2$m/sec.

Velocity head $= \dfrac{V^2}{2g} = \dfrac{2^2}{2 \times 9.8} = 0.2$ m

u s. T.E.L. = H.F.L. + velocity head

$= 306.0 + 0.20 = 306.20$ m

Head over the undersluices crest = (306.20 − 299.50)

= 6.70 m

Head over the other bays crest = (306.20 − 300.60)

= 5.60 m

As the floor and the crest of the undersluices are at the same level, the width of crest is sufficient and it will behave as broad-crested weir.

Discharge formula for broad-crested weir is given by,

$$Q = 1.705\ (L - 0.1\ n\ H)\ H^{3/2}$$

Hence, discharge passing through sluice bays, assuming that the end contractions on divide wall and abutment sides are suppressed

$= 1.705\ (75 - 0.1 \times 8 \times 6.7) \times 6.7^{1.5}$

$= 1.705 \times 69.64 \times 17.343 = 2059$ cumec

The crest width of the other bays shall be kept 2.0 m. The head over the crest is 5.60 m which is more than 1.5 times the width of crest. Thus the other barrage bays will behave as sharp crested weir.

Discharge formula for sharp crested weir :

$$Q = 1.84\ (L - 0.1\ n\ H)\ H^{3/2}$$

Hence discharge passing through weir bays

$= 1.84 \times (360 - 0.1 \times 58 \times 5.6) \times 5.6^{3/2}$

$= 1.84 \times 327.52 \times 13.252 = 7584.93$

say 7985 cumec.

Thus total discharge passing down the barrage :

$= 2059 + 7985 + 10044$

$> 10{,}000$ cumec.

Hence the assumed waterway and the crest levels are in order.

Against Lacey's 483.0 m waterway, the actual waterway of 506.0 m has beon provided.

$\therefore$ Looseness factor $= \dfrac{506.0}{483.0} = 1.05$

II. DESIGN OF UNDERSLUICES PORTION

(1) Discharge intensity and head loss under different flow conditions

(i) For maximum flood. (a) Without concentration and retrogression

Discharge intensity between piers :

$= C\,H^{3/2}$

$= 1.70 \times 6.70^{3/2}$

$= 29.48$ cumec/metre

d.s. H.F.L. $= 305.0$ m

d.s. T.E.L. $= 305.2$ m

u.s. H.F.L. (d.s. H.F.L.+afflux) $= 305 + 1\text{m} = 306$ m

u.s. T.E.L. $= 306.20$ m

Head loss (H_L) = u.s. T.E.L. − d.s. T.E.L.

$= 306.20 - 305.20 = 1.0$ m.

(b) With 20% concentration and bed retrogression by 0.5 m.

Discharge intensity $(1.20 \times 29.48) = 35.37$ cumec/metre.

Head required for this discharge intensity $= \left(\frac{35.37}{1.70}\right)^{2/3}$

$= 7.41$

u.s. T.E.L. $= 299.50 + 7.41 = 306.91$ m

d.s. H.F.L. with 0.5 m retrogression $= 304.50$ m

d.s. T.E.L. with 0.5 m retrogression $= 304.70$ m

Head loss $(H_L) = 306.91 - 304.70$ m $= 2.21$ m

(ii) Flow at Pond level (with all gates opened)

(a) Without concentration and retrogression

Head over the crest of under sluices $= 303.0 - 299.5 = 3.5$m.

Head over other bays crest $= 303.0 - 300.6 = 2.40$ m

Neglecting the velocity head for this flow condition, the total discharge passing down the barrage :

$$Q = 1.705\,(75 - 0.1 \times 8 \times 3.50)\;3.50^{3/2} + 1.84\,(3.60 - 0.1 \times 58 \times 2.40) \times 2.40^{3/2}$$

$= 806.065 + 2367.575$

$= 3173.64$ say 317.4 cumec

(2) Hydraulic Jump Calculations : *(i) Level and length of downstream floor.*

Sl. No.	Item	High flood condition: without concentration & retrogression	High flood condition: with 20% concentration and 0.5 m retrogression	Pond flow condition: without concentration & retrogression.	Pond flow condition: with 20% concentration & 0.5m retrogression
1	2	3	4	5	6
1.	Discharge intensity, q	29.48 cumec/m	35.37 cumec/m	11.60 cumec/m	13.92 cumec/m
2.	Downstream water level	305.0 m	304.50 m	302.65 m	302.15 m
3.	Upstream water level	306.0 m	306.00 m	303.00 m	303.00 m
4.	Downstream total energy level	305.20 m	304.70 m	302.75 m	302.25 m
5.	Upstream total energy level	306.20 m	306.91 m	303.10 m	303.56 m
6.	Head loss 'H_L'	1.0 m	2.21 m	0.35 m	1.31 m
7.	Downstream specific energy E_{f2} (from Fig. 2.5)	7.86 m	9.37 m	4.08 m	5.18 m
8.	Upstream specific energy E_{f1} $(E_{f1} = E_{f2} + H_L)$	8.86 m	11.58 m	4.43 m	6.49 m
9.	Level at which jump would form (d.s. T.E.L. − E_{f2})	297.34 m	295.33 m	298.67 m	297.07 m
10.	Prejump depth D_1 corresponding to E_{f1} (from Fig. 2.7)	2.71 m	2.75 m	1.60 m	1.375 m
11.	Post jump depth D_2 corresponding to E_{f2} (from Fig. 2.7)	7.0 m	8.45 m	3.50 m	4.73 m
12.	Length of concrete floor required beyond the jump $= 5\,(D_2 - D_1)$	21.45 m	28.50 m	9.50 m	16.675 m
13.	Froude no. $F = \frac{q}{\sqrt{g D_1^3}}$	2.11	2.47	1.83	2.75

Average discharge intensity $= \frac{3174}{506} = 6.3$ cumec/m.

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

$= 1.35 \left(\frac{6.3^2}{1}\right)^{1/3} = 4.66$ m.

Velocity of approach $= \frac{q}{R} = \frac{6.3}{4.66} = 1.35$ m/sec

Velocity head $= \frac{V^2}{2g} = \frac{1.35^2}{2 \times 9.8} = 0.093$ say 0.1 m

u.s. T.E.L. = 303.0 + 0.1 = 303.10 m

d.s. water level when flood discharge of 3174.0 cumec is passing = 302.65 m (from stage discharge curve fig. 5.10 where 3234 has been printed erroneously for 3174).

d.s. T.E.L. = 302.65 + 0.1 = 302.75 m.

Discharge intensity between piers $= 1.70 \times 3.60^{3/2}$

$= 11.60$ cumec/m

Head loss $(H_l) = 303.10 - 302.75 = 0.35$ m.

(ii) Plotting the prejump profiles (with the help of Fig. 2.7)

Distance from d.s. end of crest	R.L. of glacis	u.s. T.E.L.=307.10m q=35.60 cume/m High Flood F_{t1} (u.s. T.E.L. −R.L. of glacis)	D_1	u.s. T.E.L.=303.75 q=13.95 cumec/m Pond level flow E_{t1} (u.s. T.E.L. −R.L. of glacis)	D_1
m	m	m	m	m	m
3.0	298.50	8.60	3.66	5.25	1.75
4.5	298.00	9.10	3.36	5.75	1.60
7.29	297.07 (Jump location for pond flow)	10.03	3.05	6.68	1.375
9.9	296.50	10.60	2.90	—	—
12.51	295.33 (Jump location for maximum flow)	11.77	2.75	—	—

(ii) Plotting the post jump profiles (with the help of Fig. 2.8)

Sl. No.	$\frac{X}{D_1}$	High flood condition (F=2.49), F^2=6.20 and D_1 = 2.75 m Y/D_1	X	Y	Pond flow condition (F = 2.75) F^2 = 7.60 and D_1 = 1.375 m Y/D_1	X	Y
1.	1	1.3	2.75m	3.57m	1.30	1.375m	1.785m
2.	2.5	2.0	6.87m	3.57m	2.0	3.44 m	2.75 m
3.	5.0	2.5	13.75m	6.87m	2.60	6.875m	3.57 m
4.	10.15	2.8	28.0 m	7.70m	3.10	13.95m	4.27 m
5.	20.40	—	—	—	3.50	28.00m	4.82 m

(b) With 20% concentration and 0.5 m retrogression

Discharge intensity $(1.20 \times 11.60) = 13.92$ cumec/m

Head for this intensity $= \left(\frac{13.92}{1.70}\right)^{2/3} = 4.06$ m

u.s. T.E.L. = 299.50+4.06=303.56

d.s. water level with 0.5 m retrogression = 302.65−0.5 = 302.15 m

d.s. T.E.L with 0.5m retrogression = 302.15+0.1 = 302.25m

Head loss (H_l) = 303.56−302.25 = 1.31m

The downstream floor has been provided at R.L. 295.00 with a horizontal length of 28.0 m.

(3) Depth of sheet pile lines from scour considerations

(i) Depth of scour

Total discharge escaping through undersluices = 2059 cumec.

Overall waterway of undersluics = 85 metre

Average discharge intensity/metre run $= \frac{2059}{85} = 24.22$ cumec

Depth of scour 'R' $= 1.35 (q^2/f)^{1/3} = 1.35 (587/1)^{1/3}$

$= 11.07$ metre.

(ii) u.s. sheet pile

On the u.s. side allow for 1.1 R = 12.18 m

∴ R.L. of the bottom of scour hole = 306.00 − 12.18

= 293.82 m

Provide sheet pile line down to elevation 293.0 m *(iii) d.s. sheet pile*

On the downstream side allow for 1.25 R = 13.84 ≈ 14.00

R. L. of the bottom scour hole = 304.50 − 14.00 = 290.50 m

Provide sheet pile line to elevation 289.0 m.

(4) Total floor length and exit gradient

Safe exit gradient is 1/6

Maximum static head (H) = 303.0 − 295.0 = 8 m

Depth of downstream cutoff (d) = 295.00 − 289.00 = 6.0m

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

or $$\frac{1}{\pi\sqrt{\lambda}} = G_E\,(d/H) = (1/6) \times (6/8) = 0.125$$

From Khosla's exit gradient curve α = 11.50 for $\frac{1}{\pi\sqrt{\lambda}} = 0.125$

Hence requirement of total floor length (b) = αd = 11.50 × 6 = 69 m.

The floor length shall be provided as below :

Downstream horizontal floor	= 28.0 m
Downstream glacis length with 3:1 slope	= 3(299.50 − 295.00) = 13.50 m
The balance shall be provided as upstream floor	= 25.50 m
Total :	69.0 m

(5) Pressure calculations

For determining uplift pressures according to Khosla's theory it is essential to assume the floor thickness at the upstream and downstream cutoff.

Let us assume the floor thickness of 1.0 m at the upstream end, and 1.50 m at the downstream end.

(i) Upstream pile line

d = 299.50 − 293.00 = 6.50 m (including floor thickness)

$$\frac{1}{\alpha} = \frac{d}{b} = \frac{6.50}{69} = 0.095$$

$$\phi_D = 81\%$$
$$\phi_C = 73\%$$
$$\phi_D - \phi_C = 8\%$$

ϕ_C correction for depth $= \frac{1}{6.5} \times 8 = 1.23$ (+ve)

ϕ_C correction for interference of d.s. sheet pile line

d = 5.5 m, b′ = 67.50 m

D = 298.5 − 289.0 = 9.5 m, b = 69.0 m

$$C = 19\sqrt{\frac{D}{b'}} \times \frac{d+D}{b}$$
$$= 19\sqrt{\frac{9.50}{67.50}} \times \frac{5.5+9.5}{69.0}$$
$$= 1.55\%\ (+\text{ive})$$

ϕ_C corrected = 73 + 1.23 + 1.55 = 75.78% say 75.80%

(ii) Downstream pile line

d = 295.0 − 289.0 = 6.0 m

$$\frac{1}{\alpha} = \frac{d}{b} = \frac{6.0}{62} = 0.087$$

$$\phi_{E1} = 27\%$$
$$\phi_{D1} = 18\%$$
$$\phi_{E1} - \phi_{D1} = 9\%$$

ϕ_{E1} correction for depth $= \frac{1.5}{6} \times 9 = 2.25\%$ (−ve)

ϕ_{D1} correction for interference of u.s. pile line

d = 4.5 m, D = 0.5 m

$$C = 19\sqrt{\frac{0.5}{67.5}} \times \frac{4.5+0.5}{69} = 0.12\ (-\text{ve})$$

ϕ_{E1} corrected = 27 − (2.25 + 0.12) = 27 − 2.37 = 24.63% say 24.60%

(iii) The level of the hydraulic gradient lines at key points under different flow conditions are given in Table 5.3.

(6) Floor thickness

(a) Downstream floor : The hydraulic jump profile and the subsoil hydraulic gradient line can now be drawn for different flow conditions (Fig. 5.11). It would be clear from the diagram that the static head governs the thickness of the floor

Table 5.3

Condition	Down stream water level (datum) m	Up-stream water level m	Head m	Height/Elevation of subsoil H.G. line above datum					
				Upstream pile line			Downstream pile line		
				ϕ_E 100%	ϕ_D 81%	ϕ_C 75.8%	ϕ_{E1} 24.6%	ϕ_{D1} 18%	ϕ_{C1}
1	2	3	4	5	6	7	8	9	10
No flow (maximum static head)	295.0	303.0	8.0	8.0	6.48	6.06	1.97	1.44	0
				303.00	301.48	301.06	296.97	296.44	295.00
High flood	304.50	306.0	1.5	1.5	1.215	1.137	0.37	0.27	0
				306.0	305.715	305.637	304.87	304.77	302.15
Flow at pond	302.15	303.0	0.85	0.85	0.69	0.645	0.209	0.153	0
				303.0	302.84	302.795	302.359	302.303	302.15

upto 25 m from the d.s. end while beyond it the dynamic condition governs the thickness. Unbalanced heads at every 5 m along the floor have been measured and thickness calculated by dividing the unbalanced head by 1.24 which is the submerged density of concrete (2.24 − 1). For dynamic condition, 2/3rd of the value of ordinate measured between the water surface profile and the corresponding H.G. line has been taken to calculate the floor thickness.

(i) At 5.0 m from d.s. end of floor

Unbalanced head $= 2.23$ m

$\therefore$ Floor thickness required $= \dfrac{2.23}{1.24} = 1.80$ m

Provide floor thickness of 2.0 m in 5 length.

(ii) At 10.0 m from d.s. end of floor

Unbalanced head $= 2.53$ m

Floor thickness required $= \dfrac{2.53}{1.24} = 2.04$ m

Provide floor thickness of 2.25 m in 5 m length.

(iii) (iii) At 15.0 m from d.s. end of floor

Unbalanced head $= 2.83$ m

Floor thickness required $= \dfrac{2.83}{1.24} = 2.28$ m

Provide floor thickness of 2.50 m in 5 m length.

(iv) At 20.0 m from d.s. end of floor

Unbalanced head $= 3.13$ m

Floor thickness required $= \dfrac{3.13}{1.24} = 2.53$ m

Provide floor thickness of 2.75 m in 5 m length.

(v) At 25.0 m from d.s. end of floor

Unbalanced static head $= 3.42$ m

Unbalanced dynamic head $= \frac{2}{3} \times 4.65 = 3.1$ m

Floor thickness required $= \dfrac{3.42}{1.24} = 2.76$

Provide floor thickness of 3.0 m in 5 m length.

(vi) At 28.0 m from d.s. end (toe of the glacis)

Unbalanced static head $= 3.60$ m

Unbalanced dynamic head $= \frac{2}{3} \times 5.7 = 3.80$ m.

The unbalanced head due to dynamic head is more than the static head.

$\therefore$ Floor thickness required $= \frac{3.80}{1.24} = 3.06$

Provide floor thickness of 3.25 m and extend it by 2 m inside beyond the toe of glacis.

(b) Upstream floor

The subsoil hydraulic gradient line is below the water level i.e. all the unbalanced head acting on the floor is counter-balanced by the self weight of the water. Thus from theoretical considerations no floor thickness is required. However, minimum prescribed thickness of one metre shall be provided in the u.s. floor. This floor shall be thickened to 1.6 m in a length of 2.0 m under the crest.

(c) Under the glacis

The floor thickness in the sloping glacis portion shall vary from 1.60 m at the upper end to 3.25 m at the lower end,

(7) Protection works beyond impervious floor

(i) Upstream protection. *(a) Block protection*

Depth of scour 'R' $= 11.07$

Anticipated scour $= 1.5 \quad R = 16.61$ m

Upstream scour level $= 306.0 - 16.61 = 289.39$ m

Scour depth 'D' below u.s. floor $= 299.5 - 289.39$

$\doteq 10.11$ say 11.0 m

Volume of block protection to be given = 'D' cu m/m

= 11 cu m/m

Providing 1.6 m × 1.6m × 1m C.C. blocks over 0.40 m thick gravel, the length required $= \frac{11.0}{1 + 0.4} = 7.85$ m.

Provide 5 rows of the above blocks in a length of 8.0 m.

(b) Launching apron

Quantity of launching apron should be 2.25 D cu m/m.

Thickness of launching apron $= 1.40$ m

The length required $= \frac{2.25 \times 11.0}{1.40} = 17.68$ m.

Provide length of launching apron $= 18.0$ m.

(i) Downstream protection

Anticipated scour depthp $= 2\text{ R} = 22.14$

Downstream scour level $= 304.50 - 22.14 = 282.36$ m.

Scour depth 'D' below d.s. floor $= 295.00 - 282.36$

$= 14.64$ m.

(a) Inverted filler

The length of the inverted filter should be equal to 'D'

$= 14.64$ m.

Provide 1.6m × 1.6 m × 1.0 m C.C. blocks with 10 cm gap filled with 'bajri' over 1.0 m thick graded filter.

No. of rows required $= \frac{14.64}{2.00} = 7.32$.

Provide 8 rows of blocks.

(a) Launching apron

Thickness of launching apron 2.0 m

Quantity of launching apron required = 2.25D cu m/m

$\therefore$ Length required $= \frac{2.25 \times 14.64}{2.0} = 16.39$ m say 16 m

III. DESIGN OF OTHER BARRAGE BAYS PORTION

(1) Discharge Intensities and Water Levels

(i) For high flood (a) Without concentration and retrogression

u.s. T.E.L.	= 306.20 m
Crest level	= 300.60 m
Head over the crest (H)	= 306.20 − 300.60 = 5.60m
Discharge intensity	$= 1.84 \times (5.60)^{3/2}$
	= 24.38 cumec/m
u.s. H.F.L.	= 306.0 m
d.s. H.F.L.	= 305.0 m
d.s. T.E.L.	= 305.20 m
∴ Head loss (H_L)	= 306.20 − 305.20
	= 1 m

(2) Hydraulic Jump Calculations

(i) Level and Length of downstream floor.

Sl. No.	Item	High flood condition: Without concentration and retrogression	High flood condition: With 20% concentration & 0.5m retrogression.	Pond flow condition: Without concentration & retrogression	Pond flow condition: With 20% concentration & 0.5m retorgression
1.	Discharge intensity 'q'	24.38 cumec/m	29.25 cumec/m	7.27 cumec/m	8.72 cumec/m
2.	Downstream water level	305.00 m	304.50 m	302.65 m	302.15 m
3.	Upstream water level	306.00 m	306.00 m	303.10 m	303.10 m
4.	Downstream total energy level	305.20 m	304.70 m	302.75 m	302.25 m
5.	Upstream total energy level	306.20 m	306.90 m	333.10 m	303.42 m
6.	Head loss H_L	1.0 m	2.20 m	0.35 m	1.17 m
7.	Downstream specific energy E_{f2} (from Fig. 2.5)	6.95 m	8.35 m	3.05 m	3.84 m
8.	Upstream specific energy E_{f1} ($E_{f1}=E_{f2}+H_L$)	7.95 m	10.55 m	3.40 m	5.01 m
9.	Level at which jump would form (d.s. T.E.L. − E_{f2})	298.25 m	296.32 m	299.70 m	298.41 m
10.	Prejump depth D_1 corresponding to E_{f1} (from fig. 2.7)	2.29 m	2.29 m	1.065 m	1.00 m
11.	Post jump depth D_2 corresponding to E_{f2} (from fig. 2.7)	6.17 m	7.62 m	2.74 m	3.50 m
12.	Length of concrete floor required beyond the jump = $5(D_2-D_1)$	19.40 m	26.65 m	8.375 m	12.50 m
13.	Froude no. $F=(q/\sqrt{gD_1^3})$	2.24	2.69	2.12	2.80

(b) With 20% concentration and 0.5 m retrogression

Discharge intensity with 20% concentration = 1.2×24.38
= 29.25 cumec/m

Head over the crest for the discharge intensity of 29.25 cumec/m ≏ 6.3 m

∴ u.s. T.E.L. = 300.60+6.3 m
= 306.90 m

d.s. water level = 304.50 m

d.s. T.E.L. = 304.70 m

∴ Head loss (H_L) = 306.90−304.70
= 2.20 m

(ii) For pond level flow (a) Without concentration and retrogression

u.s. T.E.L. = 303.10 m

Crest level = 300.60 m

Head over the crest ≏ 2.50 m

Discharge intensity 'q' = 7.27 cumec/m

d.s. water level = 302.65 m

d.s. T.E.L. = 302.75 m

Head loss 'H_L' = 0.35 m

(iii) Plotting the prejump profiles for different flow conditions

Distance from d.s. ends of crest.	R.L. of glacis	High flood (u.s. T.E.L. with 20% concentration = 306.90) q=29.25 cumec/m E_{f1} (u.s. T.E.L. −	D_1	Pond level flow (u.s. T.E.L. with 20% concentration = 303.42) q=8.72 cumec/m E_{f1}	D_1
m	m	m	m	m	m
3.0	299.60	7.30m	3.28	3.83m	1.22m
6.07	298.41 (Jump location for pond flow	8.49m	2.75m	5.02m	1.0m
9.0	297.60	9.30m	2.51m	—	—
12.75	296.35 (Jump Location for high flood)	10.55m	2.29m	—	—

(iii) Plotting the post jump profiles for different flow conditions

Froude no. (F) for high flood condition = 2.69

Froude no. (F) for pond level flow condition = 2.80

Sl. No.	X/D_1	High flood flow condition $F^2=7.2$ $D_1=2.29$m			Flow at pond level $F^2=7.85$ $D_1=1.0$m		
		Y/D_1	X	Y	Y/D_1	X	Y
1	2	3	4	5	6	7	8
1	1.0	1.3	2.29m	2.297	1.3	1.0 m	1.3 m
2	2.5	2.0	5.75m	4.58 m	2.0	2.5 m	3.0 m
3	5.0	2.5	11.50m	5.72 m	2.6	5.0 m	2.60m
4	10.0	3.0	25.00m	6.87 m	3.2	10.0 m	3.20m
5	15.0	—	—	—	3.5	15.0 m	3.50m
6	20.0	—	—	—	3.6	20.0 m	3.60m
7	25.0	—	—	—	3.7	25.0 m	3.70m

(b) With 20% concentration and 0.5 m retrogression

Discharge intensity with 20% concentration = 8.72 cumec/m

Head over the crest for this discharge intensity = 2.82

∴ u.s. T.E.L. = (300.60+2.82)
= 303.42

d.s. water level = 302.15

d.s. T.E.L. = 302.25

Head loss 'H_L' = 303.42−302.25 = 1.17 m

The downstream floor has been provided at R.L. 296.00 m with a horizontal length of 25.0 m.

(3) Depth of sheet pile line from scour considerations

Total discharge escaping through other bays = 7985 cumec

Overall waterway of other bays = 418 m

$$\text{Average discharge intensity} = \frac{7985}{418} = 19.1 \text{ cumec/m}$$

$$\text{Depth of scour 'R'} = 1.35\left(\frac{19.1^2}{1}\right)^{1/3} = 9.46 \text{ m}$$

(i) Downstream sheet pile

On the u.s. side allow for 1.1 R = 10.4 m

∴ R.L of the bottom of scour hole = 306.00−10.4=295.6

Provide a sheet pile line up to elevation 295.00.

(ii) Downstream sheet pile

On the d.s. side allow for 1.25 R = 11.83 m

R.L. of the bottom of scour hole = 304.50−11.83 = 292.67

Provide sheet pile line down to elevation 292.00 m.

(4) Total floor length and exit gradient

Safe exit gradient is $\frac{1}{6}$

Maximum static head (H), = 303.00−296.00=7 m

Depth of downstream cutoff (d) = 296.00−292.00=4.0m

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

$$\text{Hence } \frac{1}{\pi\sqrt{\lambda}} = \frac{G_E.\,d}{H} = \frac{1}{6}\times\frac{4}{7} = 0.095$$

From Khosla's exit gradient curve, (Fig. 1.12), $\alpha = 21$

Hence total floor length (b) = αd = 21 × 4 = 84.0 m which is excessive.

Increasing the downstream cut-off by 1 m, we have

d = 5.0 m H = 7 m

$$\therefore \frac{1}{\pi\sqrt{\lambda}} = \frac{G_E.\,d}{H} = \frac{1\times 5}{6\times 7} = 0.119$$

From Khosla's curve, α = 12.5

Total floor length (b) = αd = 12.5 × 5 = 62.50 m

Adopt total floor length = 62.5 m and provide downstream cut-off to an elevation 291.0 m.

The floor length shall be provided as below :

Downstream horizontal floor = 25.0 m

Downstream glacis length with 1:3 slopes = 3(300.60−296.0)
= 13.80 m

Crest width = 2.00 m

Upstream glacis length with 1 : 1 slope = 300.60−299.50
= 1.10 m

Balance shall be provided upstream floor = 20.60 m

Total floor length $= 25.0 + 13.80 + 2.00 + 1.10 + 20.60$
$= 62.50$ m

(5) Pressure calculations

Let the floor thickness in the upstream be 1.0 m and 1.50 m near the downstream cutoff.

(i) Upstream pile line

$d = 299.50 - 295.00 = 4.50$ m

Floor length $\qquad b = 62.50$ m

$$\frac{1}{\alpha} = \frac{d}{b} = \frac{4.50}{62.50} = 0.072$$

$\phi_D = 84\%$

$\phi_C = 76\% \qquad \phi_D - \phi_C = 8\%$

ϕ_C correction for depth $= \frac{8 \times 1}{4.50} = 1.78\%$ (+ ve)

ϕ_C correction for interference of d.s. sheet pile line,

$d = 298.50 - 295.00 \qquad = 3.5$ m

$D = 298.50 - 291.00 \qquad = 7.50$ m

$b' = 61.0$ m and $b = 62.50$ m

$$C = 19\sqrt{\frac{7.5}{61}} \times \frac{11}{62.50}$$

$= 1.17\%$ (+ ve)

ϕ_C corrected $= 76 + 1.78 + 1.17 = 78.95\%$ say 79%

(ii) Downstream pile line

$d = 296.0 - 291.0 \qquad = 5.0$ m

$$\frac{1}{\alpha} = \frac{d}{b} = \frac{5.0}{62.50} = 0.08 \text{ m}$$

$\phi_{E1} = 26\%$

$\phi_{D1} = 17\% \qquad \phi_{E1} - \phi_{D1} = 9\%$

ϕ_{E1} correction for depth $= \frac{1.5}{5} \times 9 = 2.7\%$ (– ve)

ϕ_{E1} correction for interference of u.s. sheet pile line.

$d = 3.5$ m, $\qquad D = (294.25 - 295.00) = -0.75$

Hence there will be no interference of u. s. sheet pile line on the d.s. pile line.

$\therefore \phi_{E1}$ corrected $= 26.0 - 2.7 = 23.3\%$

Table 5.4

Condition	Downstream water level (datum)	Upstream water level	Head	Height/Elevation of subsoil H.G. line above datum: Upstream pile line			Downstream pile line		
	metre	metre	metre	ϕ_E 100%	ϕ_D 84%	ϕ_C 79%	ϕ_{E1} 23.3%	ϕ_{D1} 17%	ϕ_{C1} %
No flow (Maximum static head)	296.00	303.0	7.00	7.00	5.88	5.53	1.63	1.19	0
				303.00	301.88	301.53	297.63	297.19	296.00
High flood	304.50	306.00	1.50	1.50	1.26	1.185	0.35	0.255	0
				306.0	305.76	305.685	304.85	304.755	304.50
Flow at pond level	302.15	303.00	0.85	0.85	0.715	0.672	0.198	0.198	0
				303.0	302.865	302.822	302.348	302.295	302.15

(iii) The levels of the hydraulic gradient at key points under different flow conditions are given in table 5.4.

(6) Floor thickness

(a) Downstream floor *(i) At 5 m from downstream end*

Maximum unbalanced head $= 1.91$ m

Floor thickness required $= \frac{1.91}{1.24} = 1.54$ m

Provide floor thickness $= 1.75$ m

(ii) At 10 m from downstream end

Unbalanced head $= 2.22$ m

Floor thickness required $= \frac{2.22}{1.24} = 1.79$ m

Provide floor thickness $= 2.0$ m

(iii) At 15 m from downstream end

Unbalanced head $= 2.54$ m

Floor thickness required $= \frac{2.54}{1.24} = 2.05$ m

Provide floor thickness $= 2.25$ m

(iv) At 20 m from downstream end

Unbalanced head $= 2.86$ m

Floor thickness required $= \frac{2.86}{1.24} = 2.31$ m

Provide floor thickness $= 2.50$ m

(v) At 25 m from downstream end

Unbalanced head due to static condition $= 3.18$ m.

Unbalanced head due to dynamic condition

$$= \frac{2}{3} \times 5.24 = 3.49$$

Floor thickness required $= \frac{3.49}{1.24} = 2.81$

Provide floor thickness $= 3.0$ m

(b) Upstream floor :

Provide 1.0 m thick concrete floor with bottom R. L. at 298.50 and continue this upto the end of crest. The crest thickness available will be $(300.60 - 298.50) = 2.10$ m.

(c) Sloping glacis

The thickness of the glacis shall vary from 2.1 m at the upper end to 3.0 m at the lower end.

(7) Protection works beyond impervious floor

(i) Upstream protection

(b) Block protection

Depth of scour 'R' $= 9.46$ m

Anticipated scour $= 1.5$ R $= 14.2$ m

Upstream scour level $= 306.0 - 14.2$

$= 291.80$

Scour depth 'D' below u.s. floor $= 299.50 - 291.80$

$= 7.7$ m.

Quantity of block protection required = D cu m/m.

Providing 1.6 m × 1.6 m × 1.0 m C.C. blocks over 0.40 m kankar, the length required $= \frac{7.7}{1.4} = 5.5$ m

Provide 4 rows of the above blocks in a length of 6.4 m.

(b) Launching apron

Quantity of launching apron required for launch slope of 2 : 1 = 2.25 D cum/m and 1.8 D cum/m for a slope of 1.5 : 1. Assuming a slope of 1.5 : 1, the length required $= \frac{1.8 \times 7.7}{1.4}$ $= 9.9$; provide them.

(ii) Downstream protection

Expected scour = 2R $= 18.92$ m

Downstream scour level $= 304.50 - 18.92$

$= 285.58$ m

Scour depth 'D' below d.s. floor $= 296.00 - 285.58$

$= 10.42$ say 10.50 m

(a) Inverted filter

The length of inverted filter should be equal to D = 10.50 m. Provide 6 rows of 1.6m × 1.6m × 1.0m C.C. blocks with 10 cm gap in between over 1.0 m thick graded filter in a length of 10 m.

(b) Launching apron

Provide thickness of launching apron = 2.0 m

Quantity of launching apron required = 2.25 D cu. m/m.

$\therefore$ Length required $= \dfrac{2.25 \times 10.5}{2} = 11.81$ m

Provide 13 m.

(8) The detailed design is illustrated in fig. 5.11.

5.13 DESIGN EXAMPLE : HEAD REGULATOR

Design a suitable head regulator with reference to the barrage in para 5.12. The relevant data are given below :

(i) Full supply discharge of offtaking canal = 200 cumec

(ii) Full supply level of canal = 302.60 m

(iii) Water depth in canal at head = 3 m

(iv) Bed level of canal = 299.60 m

(v) Angle of the off-take of canal = 107°

(vi) Safe exit gradient for canal bed material = $\frac{1}{5}$

Draw the plan of the headworks along with the L-section of the head regulator.

Design details

(1) Fixation of crest level and waterway : Generally the crest level of the head regulator is kept 1.25 to 1.50 m higher than the crest level of the undersluices.

Adopt crest level of the regulator at R. L. 301.00 m (299.50 + 1.50).

Head regulators are generally provided with a very wide and shallow waterway. The drowned weir formula is therefore, used for calculating the discharge.

$$Q = 2/3\, c_1\, l\, \sqrt{2g}\, \{(h+h_a)^{3/2} - h_a^{3/2}\} + c_2\, l\, d\, \sqrt{2g(h + h_a)}$$

Taking $c_1 = 0.577$ and $c_2 = 0.80$, neglecting the velocity of approach 'h_a' and substituting.

$$h = 303.00 - 302.60$$
$$= 0.40 \text{ m}$$

and d = 302.60 − 301.00 = 1.60m in the above equation, we get

$$200 = (2/3) \times 0.577 \times l \times \sqrt{19.62} \times (0.40)^{3/2} + 0.8 \times l \times 1.60 \times \sqrt{19.62(0.40)}$$

$\therefore$ $l = 49.9$ metre.

Provide 6 bays of 8.50 m each giving a clear waterway of 51 m.

Provide 5 piers of 1.60 m each. Thus overall waterway of the regulator = 51+5×1.60 = 59.0 m.

(2) Hydraulic calculations for various flow conditions

(i) Full supply discharge passing down the regulator during high flood

During m-ximum flood, the u.s. water level = 306.00 m

Downstream water level in the canal for full supply discharge = 302.60 m

Head causing flow = 306.00 − 302.60 = 3.40 m

If the canal has to be fed during flood, it would be necessary to open the gates partially.

Let the gate opening be x metres ; the discharge can then be calculated with the help of the submerged orifice formula.

$$Q = CA\sqrt{2gh}$$

C = 0.62 and A = 51x. sq. metres and h = 3.40 m

$$\therefore \quad 200 = 0.62 \times 51.x \times \sqrt{19.62 \times 3.4}$$

or $x = 0.775$ m

Thus gate opening = 0.775 m

Velocity of flow through the opening $= \dfrac{200}{51 \times 0.775} = 5.06$m/sec

Loss of head at the entry $= 0.5(V^2/2g) = 0.65$m

T.E.L. just u.s. of gate = 306 + 0.2 = 306.20 m

T.E.L. just d.s. of gate = 306.20 − 0.65 = 305.55 m

Downstream water level = 302.60 m

Head loss = 305.55 − 302.60 = 2.95 m

Discharge intensity $q = \dfrac{200}{51} = 3.92$ cumec/m

(ii) Full supply discharge passing down the regulator at pond level

$$H_L = 303.00 - 302.60 = 0.40 \text{ m}$$

$$q = 3.93 \text{ cumec/m}$$

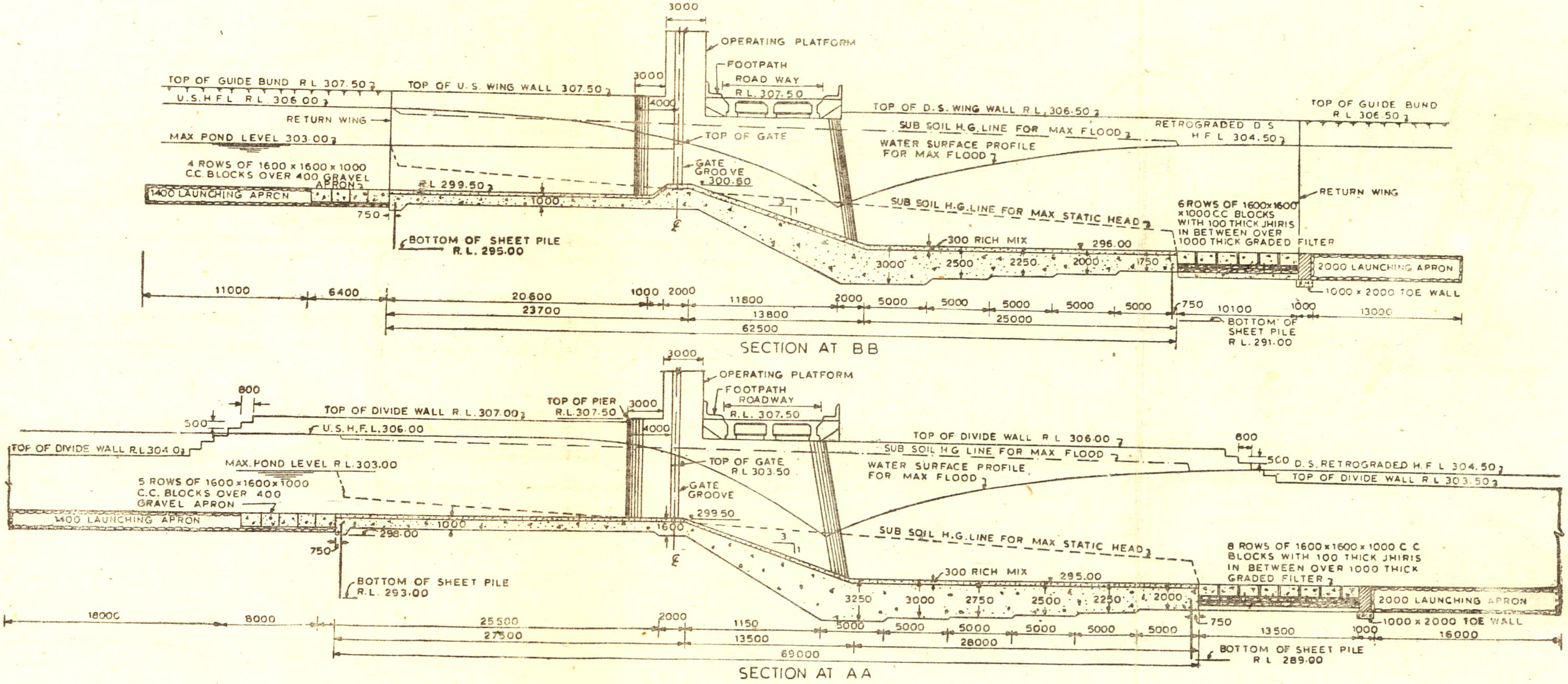


FIG. 5.11 LONGITUDINAL SECTION THROUGH UNDER SLUICE AND OTHER BAYS FLOOR

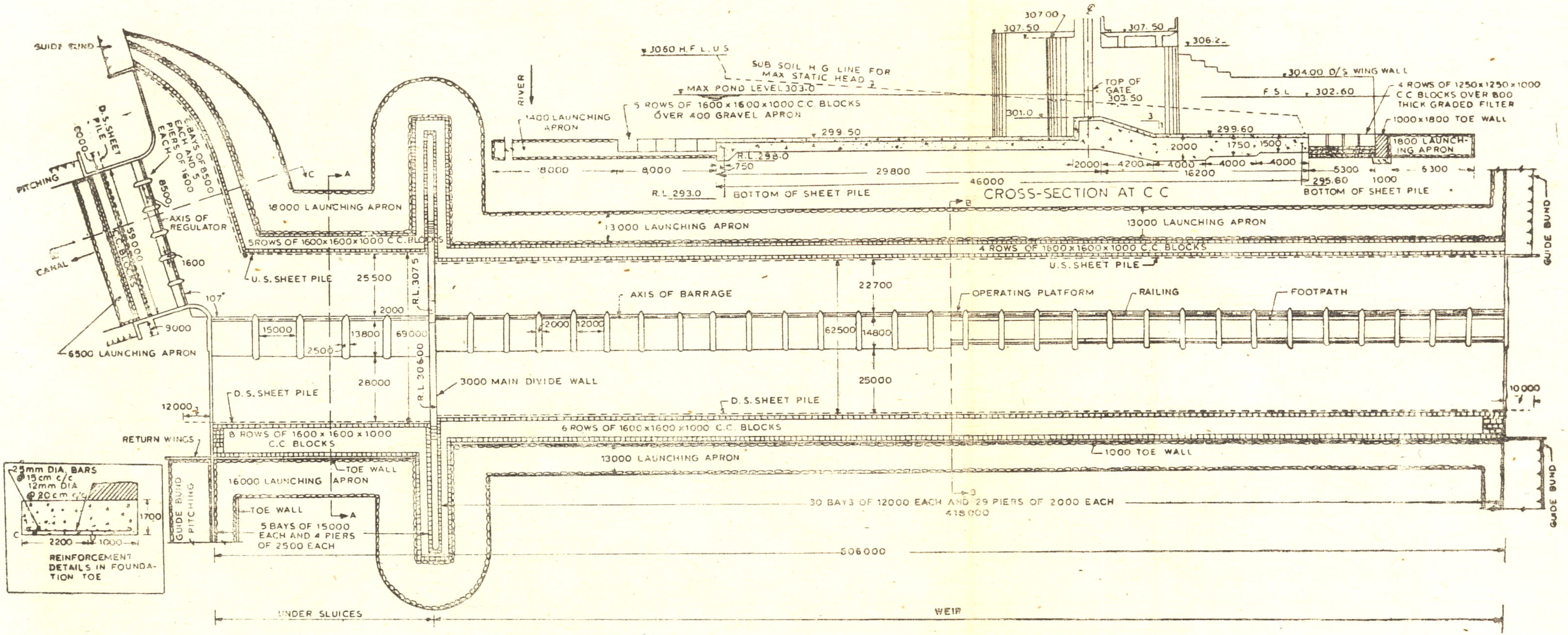


FIG. 5.12 PLAN OF HEAD WORKS