Interim Report on

Hydraulic Design Aspects of Spillway - Kalpasar Dyke

As part of

Development of Detailed Project Report of Kalpasar Spillway Project being undertaken by The National Centre for Coastal Research, Ministry of Earth Sciences

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Funded by

National Centre for Coastal Research
Ministry of Earth Sciences

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EXECUTIVE SUMMARY

INTRODUCTION

The Kalpasar Department, Government of Gujarat, proposes construction of a dam to create the fresh water coastal reservoir for irrigation, drinking and industrial purposes, in the Gulf of Khambhat region, Gujarat, India. NCCR entrusted Prof. B. S. Murty, Department of Civil Engineering, IIT Madras on the overall task of spillway design. This scope is to carry out the hydraulic, geotechnical and structural design of spillway and its associated components. Prof. V. S. Raju is the consultant for IIT Madras on this project.

The spillway is located in intertidal region of Dahej and is of 2.2 km long with 100 spans. National Center for Coastal Research (NCCR) has provided the earlier reports by the Indian Institute of Technology Roorkee for determining the Probable Maximum Flood (PMF) and length of spillway, and the report prepared by Dr. R. M. Khatsuria for preliminary design of the spillway. The location of the spillway has also been finalized by the NCCR. Spillway consists of ogee spillway, stilling basin, approach channel (from reservoir to the spillway) and spill channel (from spillway to the sea). The present interim report pertains to the Hydraulic Design of the spillway. It is accompanied by the companion interim report on Geotechnical and Structural Design Aspects of Spillway.

OVERVIEW and RESULTS

- Values for Probable Maximum Flood (PMF), the location of spillway, the length of the spillway, widths of approach and spill channels, bed levels and crest level of spillway suggested in the earlier reports have been verified and are confirmed.
- Hydraulic design for Ogee Spillway with 1:1 upstream face has been carried out and the shape of the spillway is provided.
- The cistern floor level of the stilling basin for energy dissipation on the downstream side has been determined.
- Hydraulic design of hydraulic jump type energy dissipater has been completed.
- Preliminary design for raft for spillway and the energy dissipater has been completed: (i) length of the raft and (ii) depths of u/s and downstream sheet piles to take care of scour are fixed.
- Seepage calculations have been carried out assuming the steady state seepage phenomenon through porous medium underneath a hydraulic structure. Presently, we are weighing the

options of designing the raft: Option-1: Determine thickness of raft such that the weight of raft counteracts the unbalanced uplift pressure; Option-II: Anchor the raft to piles to resist the uplift force. One of the options will be chosen based on economy and ease of construction.

- Upstream approach channel has been designed.

RECOMMENDATIONS:

1. Spillway should be of standard OGEE shape with 1:1 upstream face.
2. The floor level of the stilling basin should be at -12 m EL.
3. The length of the stilling basin = 45 m
4. The stilling basin should be of the type recommended by the Unites State Army Corps of Engineers (USACE), with two rows of baffle blocks and one end sill. Details are provided in the report.
5. The length of the raft below the spillway and the energy dissipater = 76.0 m (31 m upstream of spillway toe + 45 m downstream of spillway toe).
6. Approach channel should be of uniform width = 2200 m and a slope of S_0 = 0.0009.

TEAM

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We are thankful to Mr. S. Kiran Raju Aluri, Mrs. M. Pavithra, Mr. Madhu Sagar and Mr. Maddu Mekala of NCCR for the many in-depth discussions we had with them.

Interim Report on Hydraulic Design Aspects of Spillway

1. Preamble

The Kalpasar Department, Government of Gujarat, proposed to construct a multipurpose mega project comprising the construction of a dam for a length of about 60 km. Out of which, 30 k is in the gulf region and balance 30 km length is extended on both flanks up to nearest road crossing between Bhavnagar on the western coast and Dahej on the eastern coast of Gulf of Khambhat. The proposed spillway is located in the intertidal region of Dahej. The length of the spillway is 2.2 km. Figure 1 shows spillway location on a google map image. Borehole L1 is the only one borehole available in the entire spillway location, investigated up to a depth of 15.5 m from existing ground level (EGL) i.e., + 2.9 m by Coastal Marine Construction & Engineering Limited (COMACOE) soil investigation agency. Soil strata consist of stiff to very stiff silty clay up to 3 m followed by 3 m thick soft to very soft silty clay, and very stiff to hard silty clay up to 15.5 m. The spillway discharges surplus water from the reservoir to the sea. The width of the spillway is 2200 m with center-to-center distance between piers is 22 m. With this, there will be 100 bays. The proposed type of gate is a vertical lift gate with a width of 18 m.

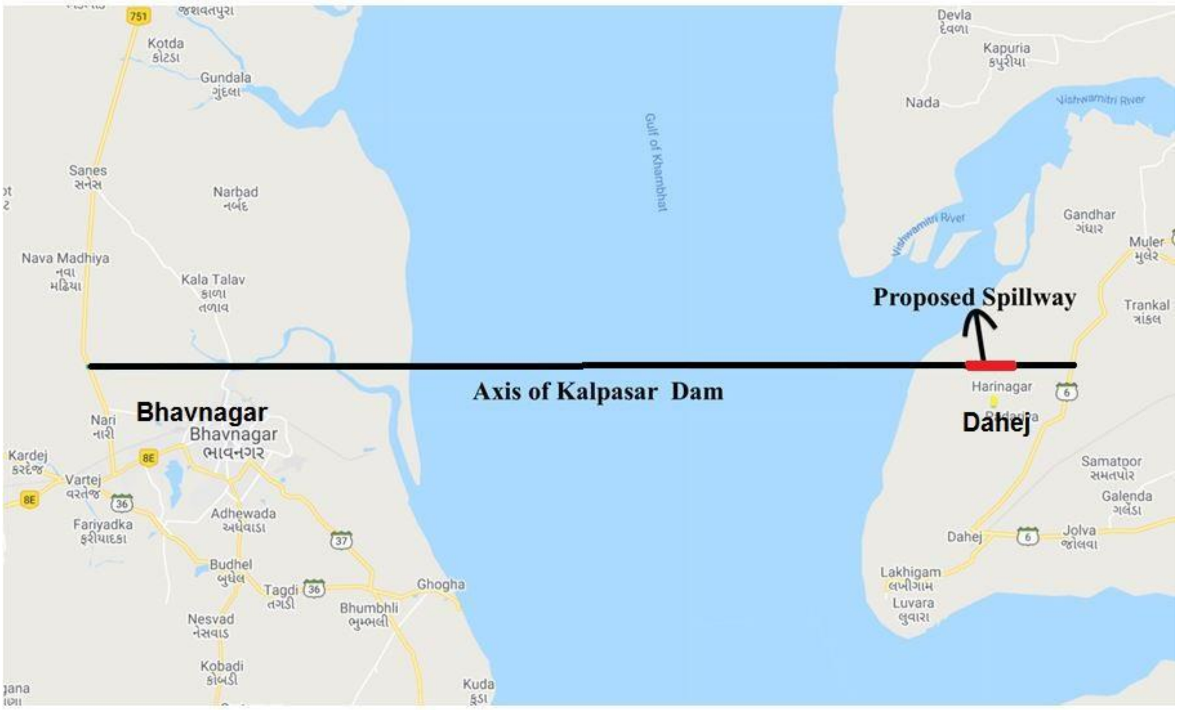


Fig. 1: Location of spillway on a google map image.

This report deals with the hydraulic design of the spillway, associated elements and the approach channel.

2. Energy Dissipation System

(i) **Type of Dissipation System:** After consideration of couple of alternatives for the energy dissipation, it was decided to go for energy dissipation using ***hydraulic jump type stilling basin***. The stilling basin will have baffle blocks and end sill for stabilizing the location for jump formation within the stilling basin for a wide range of discharge situations.

(ii) **Fixing the Cistern Floor Level & Stilling Basin Design:** One of the most important parameters in the design of hydraulic jump type stilling basin is the level of cistern floor. The floor level of cistern is fixed such that a hydraulic jump forms at the entrance to the cistern as shown in the schematic (Fig. 2) for the energy dissipater.

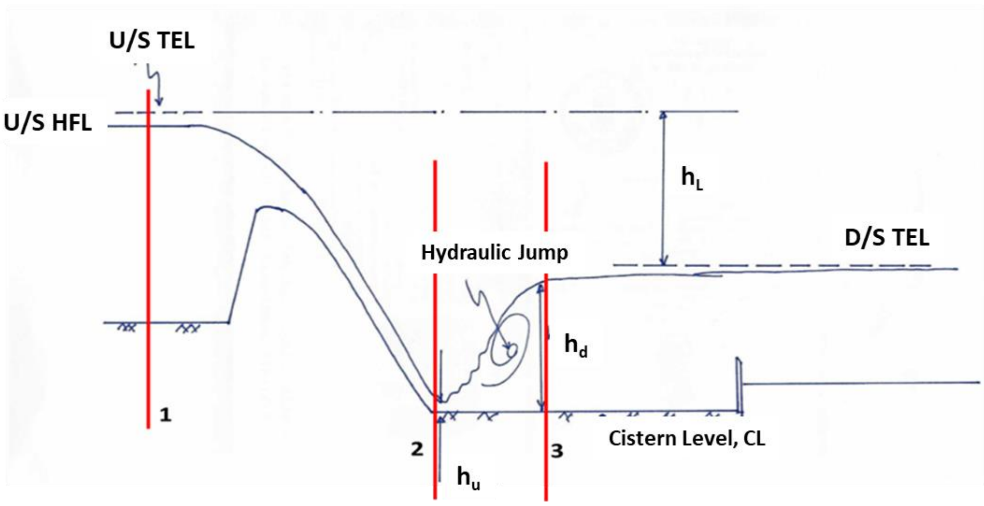


Fig. 2: Schematic for Determining the Cistern Level of Energy Dissipater

In the above figure:
USHFL = High Flood level on upstream side = + 5.0 m (all levels with respect to MSL)
DSTWL = Tail water level on downstream side
hu = Pre-jump depth
hd = Post-jump depth
CL = Cistern floor level

Applying hydraulic jump relationship between Sections-2 and 3

$$\frac{hd}{hu} = \frac{\sqrt{1+8\frac{q^2}{g.hu^3}}-1}{2} \qquad (1)$$

We assume that the total energy loss between upstream and downstream sides of spillway, hL, occurs entirely within the hydraulic jump. Head loss equation for the jump is:

$$hL = \frac{(hd-hu)^3}{4.hu.hd} \quad (2)$$

In the above equations, q = design discharge per unit width and g = acceleration due to gravity.

For given values of q and hL, equations (1) and (2) are solved simultaneously to obtain hu and hd. Then, the energy equation is applied between sections 1 and 2 to obtain the cistern level as given below.

$$USTEL = CL + hu + \frac{q^2}{2.g.hu^2} \quad (3)$$

In the above equation, USTEL = Total energy level on the upstream side of spillway.

$$USTEL = USHFL + Velocity\ head\ on\ upstream\ side \quad (4)$$

The downstream tail water level is not constant and varies continuously due to tidal variation. Therefore, while fixing the cistern floor level, we need to consider different situations.

<u>Case-1: Design the cistern for a design discharge of 110,000 m^3/s and Tail water level corresponding to lowest tide level i.e., -6.5 m MSL</u>

Design discharge = 110,000 m^3/s
Clear water way = 1760 m
Intensity of discharge, q = 110,000/1760 = 62.5 m^2/s
We add 20% to take care of flow concentrations, q = 1.2 x 62.5 = 75 m^2/s

Bed level on the upstream side of spillway = -7.0 m
HFL on the upstream side of spillway = +5.0 m
Approach flow depth = 5 + 7.0 = 12.0 m
Approach flow velocity = 75/12 = 6.25 m
Velocity head = $\frac{6.25^2}{2\times 9.81} = 2.0\ m$
USTEL = 5+2.0 = 7.0 m
DSTWL = -6.5 m
Head Loss, hL = 7+6.5 = 13.5 m

For q = 75 m^2/s and hL = 13.5 m; solution of equations (1), (2) & (3) will give:
hu = 3.15 m; hd = 17.56 m ; CL = -25.0 m

Fixing of the Cistern Floor Level at -25.0 m, while the average ground level varies from -1 to +4; could pose construction problems as well as it may lead to too much of cost.

Case-2: Design the cistern for a design discharge of 110,000 m^3/s and Tail water level corresponding to Mean Sea Level i.e., 0.0 m MSL. This is the value of downstream tail water level suggested by Mr. Khatsuria (CWPRS) in an earlier report.

Design discharge = 110,000 m^3/s
Clear water way = 1760 m
Intensity of discharge, q = 110,000/1760 = 62.5 m^2/s
We add 20% to take care of flow concentrations, q = 1.2 x 62.5 = 75 m^2/s

Bed level on the upstream side of spillway = -7.0 m
HFL on the upstream side of spillway = +5.0 m
Approach flow depth = 5 + 7.0 = 12.0 m
Approach flow velocity = 75/12 = 6.25 m
Velocity head = $\frac{6.25^2}{2\times9.81} = 2.0\ m$
USTEL = 5+2.0 = 7.0 m
DSTWL = 0.0 m
Head Loss, hL = 7+0 = 7.0 m

For q = 75 m^2/s and hL = 7.0 m; solution of equations (1) & (2) will give:
hu = 3.8 m; hd = 15.57 m ; CL = -16.7 m

Fixing of the Cistern Floor Level at -16.7 m could also pose construction problems as well as it may lead to too much of cost.

Case-3: As suggested by Mr. Khatsuria (CWPRS) in the earlier report, we may consider the design of the cistern for a design discharge of 60,000 m^3/s and Tail water level corresponding to Mean Sea Level i.e., 0.0 m MSL. It has been argued that a discharge of 110,000m^3/s corresponding to a Probable Maximum Flood will occur rarely and will last for a few hours of duration, and therefore, energy dissipater may be designed for about 50% of the design discharge value.

Design discharge = 60,000 m^3/s
Clear water way = 1760 m
Intensity of discharge, q = 60,000/1760 = 34.1 m^2/s
We add 20% to take care of flow concentrations, q = 1.2 x 34.1 = 41 m^2/s

Bed level on the upstream side of spillway = -7.0 m
HFL on the upstream side of spillway = +5.0 m
Approach flow depth = 5 + 7.0 = 12.0 m

Approach flow velocity = 41/12 = 3.4 m

Velocity head = $\frac{3.4^2}{2\times9.81} = 0.6\ m$

USTEL = 5+0.6 = 5.6 m

DSTWL = 0.0 m

Head Loss, hL = 5.6+0 = 5.6 m

For q = 41 m^2/s and hL = 5.6 m; solution of equations (1) & (2) will give:

hu = 2.41 m; hd = 10.78 m ; CL = -11.6 m

Case-4: We may consider the design of the cistern for a design discharge of 60,000 m^3/s and Tail water level corresponding to lowest Level i.e., -6.5 m MSL.

Design discharge = 60,000 m^3/s

Clear water way = 1760 m

Intensity of discharge, q = 60,000/1760 = 34.1 m^2/s

We add 20% to take care of flow concentrations, q = 1.2 x 34.1 = 41 m^2/s

Bed level on the upstream side of spillway = -7.0 m

HFL on the upstream side of spillway = +5.0 m

Approach flow depth = 5 + 7.0 = 12.0 m

Approach flow velocity = 41/12 = 3.4 m

Velocity head = $\frac{3.4^2}{2\times9.81} = 0.6\ m$

USTEL = 5+0.6 = 5.6 m

DSTWL = -6.5 m

Head Loss, hL = 5.6+6.5 = 12.1 m

For q = 41 m^2/s and hL = 12.1 m; solution of equations (1) & (2) will give:

hu = 1.92 m; hd = 12.43 m ; CL = -19.6 m

Considering all the cases, and keeping in mind the cost of energy dissipater and the challenges in construction, one can take the cistern floor level = -12.0 m, corresponding to Case 3. It may be noted that Mr. Khatsuria (CWPRS) suggested a floor level for cistern= -10.0 m.

Consequences:

During the periods when the tail water level is lower than +0.0 m, the incoming supercritical flow into the cistern will push the hydraulic jump towards the downstream side and the jump may occur downstream of cistern. This may result in erosion of sand bed of the bay. This can be controlled to a major extent by stabilizing the jump location by incorporating energy dissipating devices such as baffle blocks, end sills etc. in the ***Stilling Basin.***

Stilling Basin

A stilling basin as suggested by U. S. Army Corps of Engineers (shown in the figure 3) is adopted. Design procedure is taken from the book ***Open-Channel Flow by M. Hanif Chaudhry*** (2nd Edition, pp. 232-233).

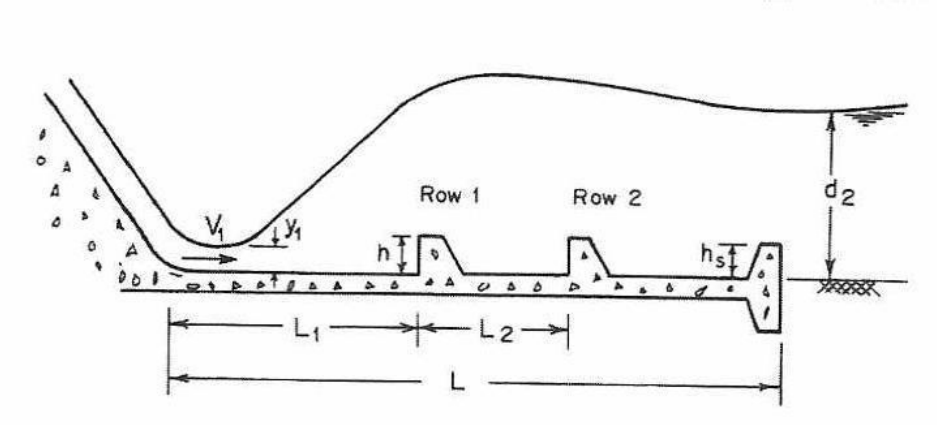


Fig. 3: Schematic of the USACE Stilling Basin

I - Option-1: Stilling Basin Floor level = -12.0 m (as per Case-3)

(A) Design Flood is assumed to be 60% of the Probable Maximum Flood (PMF)

DF = Design Flood = 0.60 x 62.5 = 37.5 m^2/s

Standard Project Flood is assumed to be only 80% of the Design Flood

SPF = Standard Project Flood = 0.80 x 37.5 = 30.0 m^2/s

Assuming energy loss on spillway face = 5% of total head corresponding to PMF Head loss = 0.05 x 13.5 = 0.7 m

Approach velocity = 37.5/12 = 3.125 m/s

TEL on the upstream side = $+5 + \frac{3.1^2}{2 \times 9.81} = 5.5\ m$

TEL at the entrance to the stilling basin = TEL1= +5.5 – 0.7 = 4.8 m

$$TEL1 = CL + y1 + \frac{q^2}{2.0.g.y1^2}$$

Corresponding to Design Flood;

$4.8 = -12.0 + y1 + \frac{37.5^2}{2.0.g.y1^2}$

Y1 = 2.22 m

V1 = 16.9 m/s

Froude Number = F1 = $\frac{V1}{\sqrt{g.y1}} = 3.62$

Sequent depth y2 = $y1 \times \left(\frac{\sqrt{1+8F1^2}-1}{2}\right) = 10.31\ m$

Corresponding to Standard Project Flood;

$$4.8 = -12.0 + y1 + \frac{30^2}{2.0.g.y1^2}$$

Y1' = 1.75 m

V1' = 17.1 m/s

Froude Number = F1' = $\frac{V1}{\sqrt{g.y1}} = 4.12$

Sequent depth y2' = $y1 \times \left(\frac{\sqrt{1+8F1^2}-1}{2}\right) = 9.4\ m$

F1 < 4.6

L1 = 1.5 x y2 = 15.5 m	**L1 = 16.0 m**
h = y2/6 = 10.31/6 = 1.7 m;	**h = 1.7 m**
L2 = 2.5 x h = 4.3 m	**L2 = 4.3 m**
hs = h/2 = h/2 = 0.9 m	**hs = 1.0 m**
d2 = approximately y2' = 9.4 m	
d2 > 0.85 x y2 = 8.8 m	**d2 = (approximately) 9.0 m**
L > L1 + y2 > 16 + 10.3 > 26.3	
L > 4 x 10.3 > 41.2 m	**L = 45 m**

Stagger the baffle blocks in rows 1 and 2.

Width of Baffle block = h = 1.7 m

Spacing between baffle blocks = 1.7 m

Downstream water level = -12+9.0 = -3.0

This means, the hydraulic jump will form at the entrance to stilling basin when tail water level is -3.0 m. The lowest tail water level is -6.5 m. For tail water levels lower than -3.0 m, the hydraulic jump will have a tendency to move downstream. Downstream protection of bed is provided to take care of this. For tail water levels higher than -3.0 m; jump will move upstream and may get submerged if the TWL is very high. It will form on the sloping face of spillway.

(B) Design Flood is assumed to be 70% of the Probable Maximum Flood (PMF)

DF = Design Flood = 0.70 x 62.5 = 43.8 m^2/s

Standard Project Flood is assumed to be only 80% of the Design Flood

SPF = Standard Project Flood = 0.80 x 43.8 = 35.0 m^2/s

Assuming energy loss on spillway face = 5% of total head corresponding to PMF

Head loss = 0.05 x 13.5 = 0.7 m

Approach velocity = 43.8/12 = 3.65 m/s

TEL on the upstream side = $+5 + \frac{3.65^2}{2\times 9.81} = 5.7\ m$

TEL at the entrance to the stilling basin = TEL1= +5.7 – 0.7 = 5.0 m

$$TEL1 = CL + y1 + \frac{q^2}{2.0.g.y1^2}$$

<u>Corresponding to Design Flood;</u>

$$5.0 = -12.0 + y1 + \frac{43.8^2}{2.0.g.y1^2}$$

Y1 = 2.60 m

V1 = 16.8 m/s

Froude Number = F1 = $\frac{V1}{\sqrt{g.y1}} = 3.32$

Sequent depth y2 = $y1 \times \left(\frac{\sqrt{1+8F1^2}-1}{2}\right) = 11.0\ m$

<u>Corresponding to Standard Project Flood;</u>

$$5.0 = -12.0 + y1 + \frac{35^2}{2.0.g.y1^2}$$

Y1' = 2.04 m

V1' = 17.1 m/s

Froude Number = F1' = $\frac{V1}{\sqrt{g.y1}} = 3.82$

Sequent depth y2' = $y1 \times \left(\frac{\sqrt{1+8F1^2}-1}{2}\right) = 10.1\ m$

F1 < 4.6

L1 = 1.5 x y2 = 16.5 m	**L1 = 16.5 m**
h = y2/6 = 11/6 = 1.8 m;	**h = 1.8 m**
L2 = 2.5 x h = 4.6 m	**L2 = 4.6 m**
hs = h/2 = h/2 = 0.9 m	**hs = 1.0 m**
d2 = approximately y2' = 10.1 m	
d2 > 0.85 x y2 = 9.4 m	**d2 = (approximately) 9.7 m**
L > L1 + y2 > 16.5 + 11.0 > 27.5	
L > 4 x 11.0 > 44.0 m	**L = 45 m**

Stagger the baffle blocks in rows 1 and 2.

Width of Baffle block = h = 1.8 m

Spacing between baffle blocks = 1.8 m

Downstream water level = -12+9.7 = -2.3

<u>This means, the hydraulic jump will form at the entrance to stilling basin when tail water level is -2.3 m. The lowest tail water level is -6.5 m. For tail water levels lower than -2.3 m, the hydraulic jump will have a tendency to move downstream.</u>

II - Option-2: Stilling Basin Floor level = -10.0 m (as per suggestion by Mr. Khatsuria, CWPRS)

(A) Design Flood is assumed to be 60% of the Probable Maximum Flood (PMF)

DF = Design Flood = 0.60 x 62.5 = 37.5 m^2/s

Standard Project Flood is assumed to be only 80% of the Design Flood

SPF = Standard Project Flood = 0.80 x 37.5 = 30.0 m^2/s

Assuming energy loss on spillway face = 5% of total head corresponding to PMF

Head loss = 0.05 x 13.5 = 0.7 m

Approach velocity = 37.5/12 = 3.125 m/s

TEL on the upstream side = $+5 + \frac{3.1^2}{2 \times 9.81} = 5.5\ m$

TEL at the entrance to the stilling basin = TEL1= +5.5 – 0.7 = 4.8 m

$$TEL1 = CL + y1 + \frac{q^2}{2.0.g.y1^2}$$

Corresponding to Design Flood;

$$4.8 = -10.0 + y1 + \frac{37.5^2}{2.0.g.y1^2}$$

Y1 = 2.40 m

V1 = 15.6 m/s

Froude Number = F1 = $\frac{V1}{\sqrt{g.y1}} = 3.2$

Sequent depth y2 = $y1 \times \left(\frac{\sqrt{1+8F1^2}-1}{2}\right) = 9.8\ m$

Corresponding to Standard Project Flood;

$$4.8 = -10.0 + y1 + \frac{30^2}{2.0.g.y1^2}$$

Y1' = 1.9 m

V1' = 15.9 m/s

Froude Number = F1' = $\frac{V1}{\sqrt{g.y1}} = 3.68$

Sequent depth y2' = $y1 \times \left(\frac{\sqrt{1+8F1^2}-1}{2}\right) = 9.0\ m$

F1 < 4.6

L1 = 1.5 x y2 = 14.7 m	**L1 = 15.0 m**
h = y2/6 = 9.8/6 = 1.6 m;	**h = 1.6 m**
L2 = 2.5 x h = 4.0 m	**L2 = 4.0 m**

hs = h/2 = h/2 = 0.8 m	**hs = 0.8 m**
d2 = approximately y2' = 9.0 m	
d2 > 0.85 x y2 = 8.3 m	**d2 = (approximately) 8.7 m**
L > L1 + y2 > 15 + 9.8 > 24.8	
L > 4 x 9.8 > 39.2 m	**L = 40 m**

Stagger the baffle blocks in rows 1 and 2.

Width of Baffle block = h = 1.6 m

Spacing between baffle blocks = 1.6 m

Downstream water level = -10+8.7 = -1.3

This means, the hydraulic jump will form at the entrance to stilling basin when tail water level is -1.3 m. The lowest tail water level is -6.5 m. For tail water levels lower than -1.3 m, the hydraulic jump will have a tendency to move downstream.

(B) Design Flood is assumed to be 70% of the Probable Maximum Flood (PMF)

DF = Design Flood = 0.70 x 62.5 = 43.8 m^2/s

Standard Project Flood is assumed to be only 80% of the Design Flood

SPF = Standard Project Flood = 0.80 x 43.8 = 35.0 m^2/s

Assuming energy loss on spillway face = 5% of total head corresponding to PMF

Head loss = 0.05 x 13.5 = 0.7 m

Approach velocity = 43.8/12 = 3.65 m/s

TEL on the upstream side = $+5 + \frac{3.65^2}{2\times 9.81} = 5.7\ m$

TEL at the entrance to the stilling basin = TEL1= +5.7 – 0.7 = 5.0 m

$$TEL1 = CL + y1 + \frac{q^2}{2.0.g.y1^2}$$

Corresponding to Design Flood;

$$5.0 = -10.0 + y1 + \frac{43.8^2}{2.0.g.y1^2}$$

Y1 = 2.84 m

V1 = 15.4 m/s

Froude Number = F1 = $\frac{V1}{\sqrt{g.y1}} = 2.92$

Sequent depth y2 = $y1 \times \left(\frac{\sqrt{1+8F1^2}-1}{2}\right) = 10.4\ m$

Corresponding to Standard Project Flood;

$$5.0 = -10.0 + y1 + \frac{35^2}{2.0.g.y1^2}$$

Y1' = 2.21 m

V1' = 15.8 m/s

Froude Number = F1' = $\frac{V1}{\sqrt{g.y1}} = 3.4$

Sequent depth y2' = $y1 \times \left(\frac{\sqrt{1+8F1^2}-1}{2}\right) = 9.6\ m$

F1 < 4.6

L1 = 1.5 x y2 = 15.6 m	**L1 = 15.6 m**
h = y2/6 = 10.4/6 = 1.7 m;	**h = 1.7 m**
L2 = 2.5 x h = 4.3 m	**L2 = 4.3 m**
hs = h/2 = h/2 = 0.85 m	**hs = 1.0 m**
d2 = approximately y2' = 9.6 m	
d2 > 0.85 x y2 = 8.8 m	**d2 = (approximately) 9.2 m**
L > L1 + y2 > 15.6 + 10.4 > 26.0	
L > 4 x 10.4 > 42.0 m	**L = 45 m**

Stagger the baffle blocks in rows 1 and 2.

Width of Baffle block = h = 1.7 m

Spacing between baffle blocks = 1.7 m

Downstream water level = -10+9.2 = -0.8

This means, the hydraulic jump will form at the entrance to stilling basin when tail water level is -0.8 m. The lowest tail water level is -6.5 m. For tail water levels lower than -0.8 m, the hydraulic jump will have a tendency to move downstream.

Considering the stabilization of jump within the stilling basin for large range of flow discharges and tail water level conditions: following are recommended for stilling basin dimensions and levels.

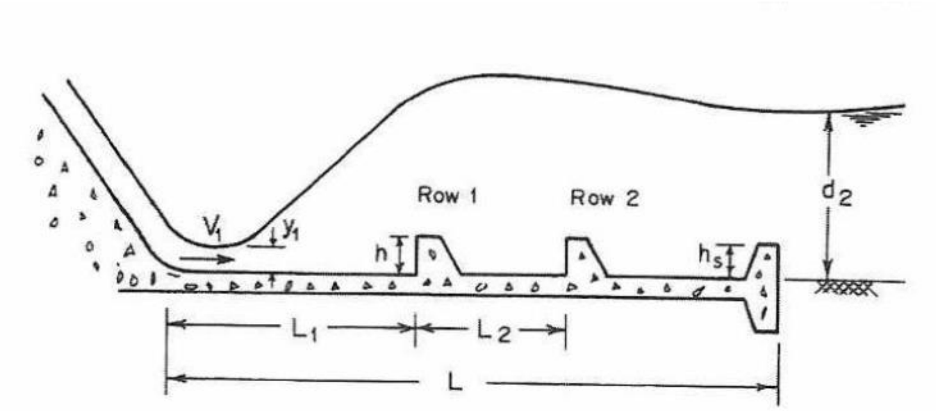


Fig. 4: Schematic of the USACE Stilling Basin

Floor level of stilling basin (cistern): -12.0 m;

L1 = 16.0 m

h = 1.7 m

L2 = 4.3 m

hs = 1.0 m

L = 45 m

Stagger the baffle blocks in rows 1 and 2.

Width of Baffle block = h = 1.7 m

Spacing between baffle blocks = 1.7 m

3. Spillway Dimensions

Length of the Spillway:

Crest Level of Spillway = -3.5 m
HFL = +5.0 m
Design Head: Hd = 5+3.5 = 8.5 m
Bed Level of Upstream Channel = -7.0 m
Height of Spillway = P = 7.0-3.5 = 3.5 m
Approach Velocity =$\frac{110{,}000}{12\times2200} = 4.2\,\frac{m}{s}$
Velocity Head = 0.9 m
Design Energy Head = He = 8.5 + 0.9 = 9.4 m
P/He = 0.37
For this value of P/He; Cd = 0.705 (if upstream face = vertical)
(Fig. 7-20 in Chaudhry's book)
Applying correction for sloping upstream face; (From Vente Chow's book; Fig. 14.4)
P/Hd = 3.5/8.5 = 0.41; Correction factor = 1.04; Cd = 0.733

Cd value gets reduced when there is submergence i.e., d/s water level is higher than the crest level even if the u/s water level is higher than the d/s water level.

Downstream water level is calculated as follows

DF = Design Flood = 62.5 m^2/s

Assuming energy loss on spillway face = 5% of total head corresponding to PMF Head loss = 0.05 x 13.5 = 0.7 m

Approach velocity = 62.5/12 = 5.2 m/s

TEL on the upstream side = $+5 + \frac{5.2^2}{2\times 9.81} = 6.38\ m$

TEL at the entrance to the stilling basin = TEL1= +5.5 – 0.7 = 5.7 m

$$TEL1 = CL + y1 + \frac{q^2}{2.0.g.y1^2}$$

Corresponding to Design Flood;

$5.7 = -12.0 + y1 + \frac{62.5^2}{2.0.g.y1^2}$

Y1 = 3.78 m

V1 = 16.5 m/s

Froude Number = F1 = $\frac{V1}{\sqrt{g.y1}} = 2.71$

Sequent depth y2 = $y1 \times \left(\frac{\sqrt{1+8F1^2}-1}{2}\right) = 12.7\ m$

Water level on d/s side = -12+12.7 = +0.7 m

d = 12.7 m

hd = 5-0.7 = 4.3

hd+d = +5 – (-12) = 17.0 m

Hd = 8.5 m

(hd+d)/Hd = 17/8.5 = 2.0

hd/Hd = 4.3/8.5 = 0.5

Percentage reduction = 0.8%

Cd = 0.733

Intensity of discharge, q = $\frac{2}{3} cd \times \sqrt{2.g} He^{1.5}$ =62.4 m²/s

Effective length of spillway = 1762 m

Number of bays Provided = 100

Width of gate = 18 m

Clear water way = 1800 m

Number of Piers = 99

Actual head on crest = 9.4 m

Kp = pier contraction coefficient = 0.02

Ka = abutment contraction coefficient = 0.1

Leff = L – 2(N.kp+ka)xHe = 1761 m

Spillway Length: Provide 100 bays; Width of each bay = 18 m

Spillway Shape

Spillway shape is shown in Figure 5. In the present case we adopt an upstream face with a slope of 1:1. The downstream face i.e., to the right of origin O; the equation for face is given as:

$$\frac{y}{Hd} = K\left[\frac{x}{Hd}\right]^n$$

Where for an upstream face with a 1:1 slope; K = 0.534 and n = 1.776. In the present case Hd = Design upstream head = 9.4 m. Therefore; equation for downstream face of spillway:

$$\boldsymbol{X^{1.776} = 10.66 \times Y}$$

This curve joins smoothly to the cistern bed level at -12.0 m. That is maximum value of y = 8.5 m.

The equation for the upstream portion of the spillway is as follows: (Subramanya: Open-Channel Flow)

$$\frac{y}{Hd} = 0.724 \times \left[\frac{x}{Hd} + 0.270\right]^{1.85} - 0.432 \times \left[\frac{x}{Hd} + 0.270\right]^{0.625} + 0.126$$

This equation is valid in the region

$$0 \geq \frac{x}{Hd} \geq -0.270 \quad \& \quad 0 \leq \frac{y}{Hd} \leq 0.126$$

Upstream curve should join smoothly with the 1:1 sloped upstream face

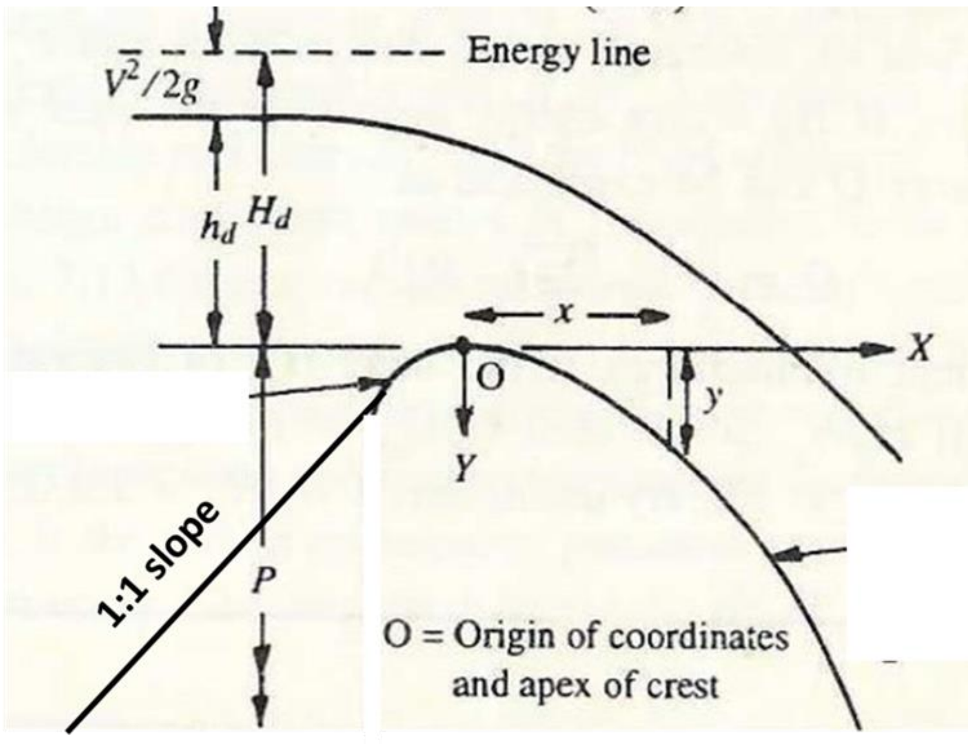


Fig. 5: Schematic of Profile of Spillway

4. Spillway Raft & Cutoff Piles:

There is a concrete raft on which spillway and the energy dissipater will be placed as shown in Fig. 6.

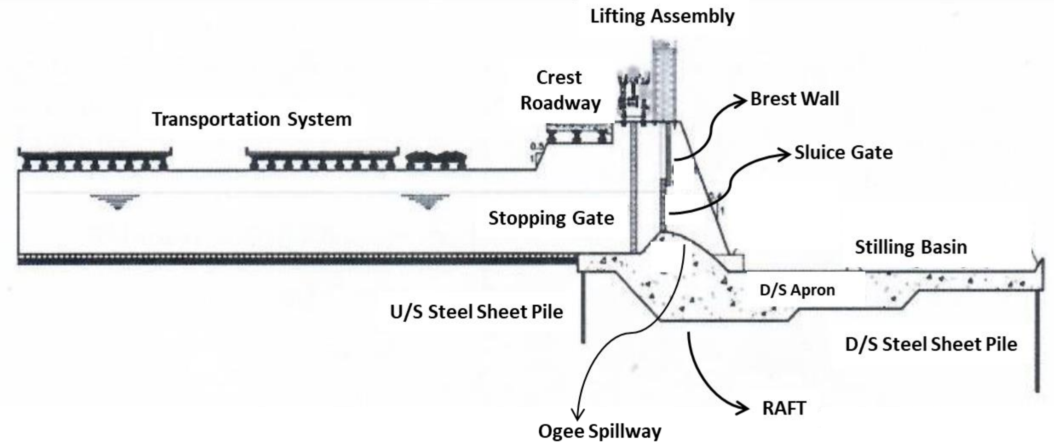


Fig. 6: Cross Section of Spillway and Energy Dissipater with Concrete Raft

Typically this raft is designed based on the design concepts for hydraulic structures on permeable foundations i.e., hydraulic structures in rivers on alluvial soils. The raft length is based on the calculations for exit gradient for seepage flow. Sizes of sheet piles are based on scour phenomenon due to surface flow condition. The thickness of raft at different locations is based on the unbalanced pressure due to (i) uplift pressure exerted by seepage flow and (ii) downward pressure due to the water depth above the structure.

At this stage, we are suggesting these assuming that the soil is cohesionless. However, it is known that conditions are clayey at the site. Therefore, suggested design is safe but it may be conservative and also may encounter difficulties during construction. We are studying this in depth and will revise the sizes before finalizing the DPR.

(a) Sizes of Sheet Piles (Based on Scour Calculations)

Maximum flood Discharge = 120,000 m^3/s (Q)

Lacey's Regime width = $4.83 \times \sqrt{Q}$ = 1673 m (A total width of 2200 m is provided)

Looseness Factor = $^{2200}/_{1673} = 1.3$

Depth of Scour below HFL = $= 0.473 \left(\frac{Q}{f}\right)^{0.33}$

f = Silt factor $= 1.76\sqrt{mr}$; mr = average particle size in mm = 0.34 mm

f = 1.01

Depth of Scour below HFL = $= 0.473\left(\frac{120{,}000}{1.01}\right)^{0.33} = 23.2\ m$

U/S Cutoff Size: As per the IS Code-6966; U/S cutoff should be equal to 1R

U/S Water level = +5.0 EL
U/S Floor Level = -7.10 EL
Scour Depth = 1R = 23.2 m
Bottom level of U/S Cutoff = -18.2 m
Depth of U/S Cutoff = 18.2 -7.1 = 11.1 m
Suggested size of U/S Sheet Pile for cutoff = 11.5 m

D/S Cutoff Size: As per the IS Code-6966; D/S cutoff should be equal to 1.25R

D/S Water level = -3.0 EL
D/S Floor Level = -12.0 EL
Scour Depth = 1.25R = 29 m
Bottom level of D/S Cutoff = -32 m
Depth of D/S Cutoff = 32 -12 = 20 m
Suggested size of D/S Sheet Pile for cutoff = 20 m

Alternative:
We are also showing the calculations below for 60% of PMF, for which the stilling basin is designed. Also, the scour depth calculated for cohesionless soils may not be reailized for cohesive soils.

R = 19.6 m
Bottom Level of U/S Cutoff = -14.6 m
Depth of U/S Cutoff = 14.6 – 7.1 = 7.5 m (to be confirmed in the final DPR)

Bottom Level of D/S Cutoff = -27.5 m
Depth of D/S Cutoff = 27.5 – 12.0 = 15.5 m (to be confirmed in the final DPR)

(b) Length of the Raft (Based on Exit Gradient)

Specific Gravity of Soil, S = 2.60
Porosity, n = 0.4
Exit Gradient, $G_e = \frac{H}{d}\frac{1}{\pi\sqrt{\lambda}}$ (Schematic is shown in Fig. 6)

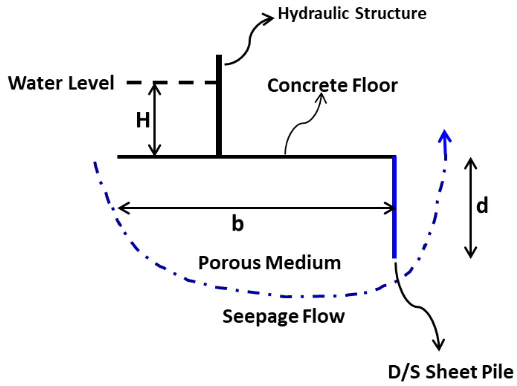


Fig. 6: Schematic for Calculating Exit Gradient

Critical Exit Gradient = = $(S-1)*(1-n) = 0.96$

Assuming a Factor of safety of 7; Safe Exit Gradient = 0.142

$G_e = \frac{H}{d}\frac{1}{\pi\sqrt{\lambda}} = 0.142$ $\lambda = \frac{1+\sqrt{1+\alpha^2}}{2}$ $\alpha = \frac{b}{d}$

d = depth of D/S sheet Pile = 15.5 m

Maximum Head causing the flow = MWL – Lowest Tail Water Level = +5.0 – (- 6.5) = 11.5 m

$\lambda = 2.8$ $\alpha = 4.4;$ $b = 69\,m$

Provide a Floor Length of 76 m

Minimum Floor Length Required on D/S side = 45 m

Rest 31.0 m is provided on U/S of toe of the Ogee Spillway.

(c) Uplift Pressures (based on Khosla's theory):

At this stage, we are assuming that Khosla's theory for seepage flow through permeable medium under hydraulic structures is valid in the case of clayey soils and temporally varying water level conditions both on u/s and d/s sides.

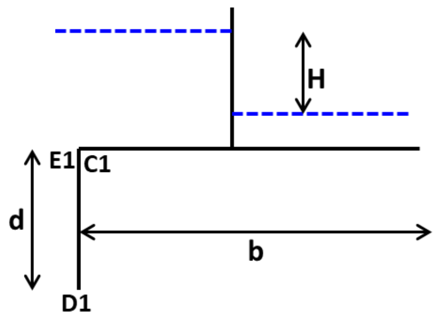
Two possible critical situations:

(a) Water level is at +3.0 (FRL); there is no flow over the spillway and the Tail water level is at its minimum i.e., at -6.5 m. Head causing the flow = 9.5 m

(b) Water level is at +5.0 (MFL); there is flow corresponding to PMF over the spillway and the tail water level is at its minimum i.e., at -6.5 m. Supercritical flow occurs in stilling basin with a depth equal to 3.9 m (y). The hydraulic jump shoots out of stilling basin.

$$5.0 = -12.0 + y + \frac{62.5^2}{2.0.g.y1^2}$$

Uplift Calculations for KEY POINTS for U/S Piles



Total length of floor = b = 76 m

Depth of U/S Pile line below floor = d = 11.5 m

$$\alpha = \frac{b}{d} = 6.6$$

$\phi_{E1} = 100\%$
$\phi_{C1} = 65.9\%$
$\phi_{D1} = 76.5\%$

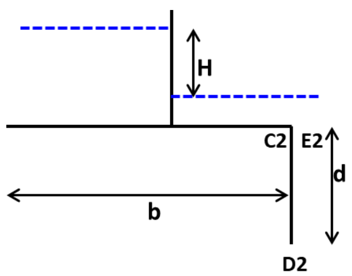
Correction for Mutual interference: = $19\sqrt{\frac{(32-7.1)}{75.0}}.\frac{24.9+11.5}{76} = +5.3\%$

Correction for Thickness (Assume thickness = 3.0 m):

$$= \frac{76.5 - 65.9}{11.5} \times 3 = 2.76\%$$

$\boldsymbol{\phi_{E1} = 100\%}$
$\boldsymbol{\phi_{C1} = 65.9 + 5.3 + 2.76 = 74\%}$
$\boldsymbol{\phi_{D1} = 76.5\%}$

Uplift Calculations for KEY POINTS for D/S Piles



Total length of floor = b = 76 m

Depth of D/S Pile line below floor = d = 20 m

$$\alpha = \frac{b}{d} = 3.8$$

$\phi_{C2} = 44\%$
$\phi_{D2} = 30\%$
$\phi_{E2} = 0\%$

Correction for Mutual interference: = $-19\sqrt{\frac{11.5}{75.0}}.\frac{24.9+11.5}{76} = -3.5\%$

Correction for Thickness (Assume thickness = 3.0 m):

$$= -\frac{44-30}{20} \times 3 = -3.6\%$$

$\boldsymbol{\phi_{E2} = 0\%}$
$\boldsymbol{\phi_{C2} = 44 - 3.5 - 3.6 = 37\%}$
$\boldsymbol{\phi_{D2} = 30\%}$

Uplift pressure is assumed to vary linearly from Point C1 to C2 under the floor.

<u>Unbalanced pressure head at Key Points for critical case (a): (Fig. 7)</u>

$$h_{C1} = (HGL\ due\ to\ seepage - Floor\ level) - Surface\ water\ depth$$

$$h_{C1} = [(0.74 \times 9.5 + (-6.5)) - (-7.0)] - 10.0 = -2.47\ m$$

$$h_{C2} = [(0.37 \times 9.5 + (-6.5)) - (-12.0)] - 5.5 = 3.52\ m$$

$$h_{C3} = [-0.93 + 12] - 5.5 = 5.57\ m$$

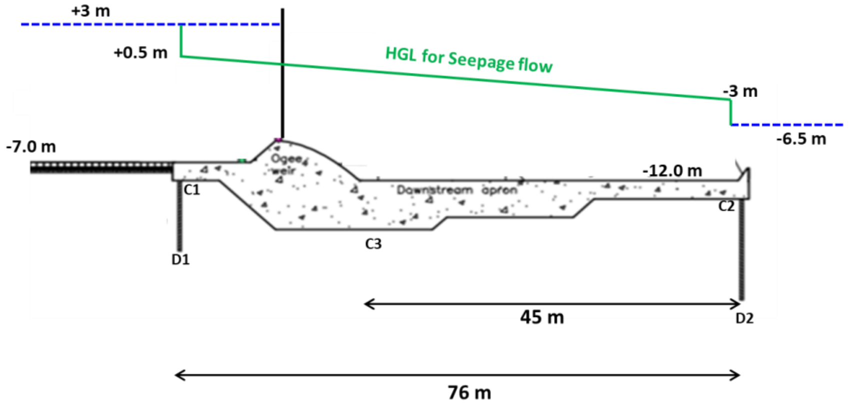


Fig. 7: HGL for Seepage Line for Critical Case (a) (not to scale)

Unbalanced pressure head at Key Points for critical case (b): (Fig. 8)

$$h_{C1} = (HGL\ due\ to\ seepage - Floor\ level) - Surface\ water\ depth$$

$$h_{C1} = [(0.74 \times 13.1 + (-8.1)) - (-7.0)] - 12.0 = -3.4\ m$$

$$h_{C2} = [(0.37 \times 13.1 + (-8.1)) - (-12.0)] - 3.9 = 4.9\ m$$

$$h_{C3} = [-0.35 + 12] - 3.9 = 7.75\ m$$

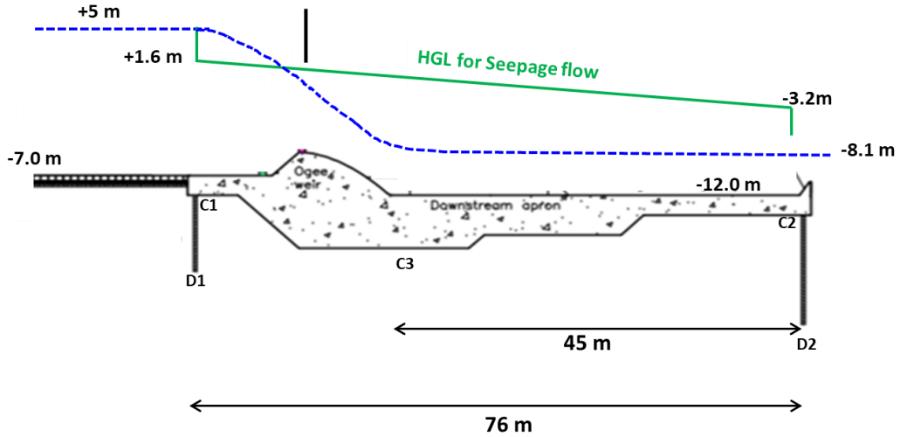


Fig. 8: HGL for Seepage Line for Critical Case (b) (not to scale)

Critical values of unbalanced head

@ The toe of spillway = 8.0 m

@ D/s end of Stilling basin = 4.9 m

The thickness of floor should vary from 8.0/1.4 = 5.7 m to 3.5 m to counter this unbalanced uplift pressure.

5. Longitudinal Slope of the Approach Channel

Schematic for flow in approach channel is shown in Fig. 9.

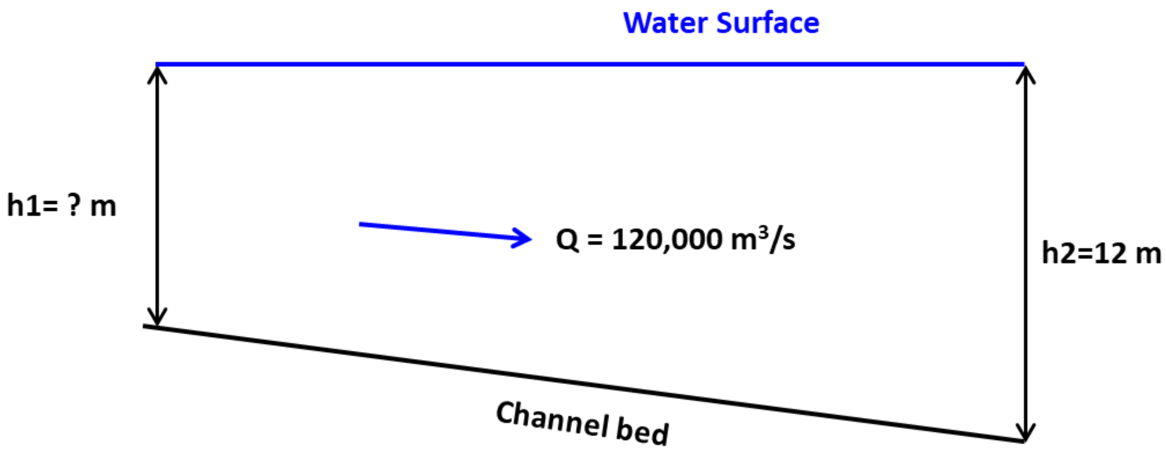


Fig. 9: Schematic for flow in approach channel

Applying energy equation between Sections 1 and 2

$$h1 + Z1 + \frac{q^2}{2.g.h1^2} = h2 + Z2 + \frac{q^2}{2.g.h2^2} + \frac{q^2.n^2}{h2^{3.333}} \times L$$

q = Q/B = 120,000/2200 = 54.5 m^2/s

h2 = +5 – (-7)=12 m

<u>Assuming a representative value of n = 0.014</u>

L = Length of approach channel = 2000 m

Foe level water surface; h1+Z1 = h2+Z2

H1 = 10.6 m; H1 = 10.2 m if corrected for variation in friction slope from entrance of approach channel to the spillway

Bed slope = (12-10.2)/2000 = 0.0009

The velocity in the channel varies from 5.4 m/s to 4.6 m/s

6. Closure

The design will be finalized based on the discussions in the progress review meeting and finalization of few parameters such as roughness coefficient. Design of raft will also be finalized in the next one month based on detailed calculations for seepage flow. Design for the downstream spill channel and extra protection works for protecting the channels will also be finalized in the next one month