Flood Regulator of Kalpasar Dyke

Detailed Project Report

As part of

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Flood Regulator Design

Flood Regulator is a structure constructed in dyke to discharge flood water from the reservoir to the sea and avoids inundation in the adjacent areas of the reservoir. It also protects against the intrusion of seawater into the reservoir side especially, during high tides and storm surges thereby avoiding coastal flooding.

15.1 Past Studies

The Department of Hydrology, IIT Roorkee estimated the Design flood PMF (Probable Maximum Flood) and Capacity of the Flood Regulator for the Kalpasar project. Accordingly, a preliminary study was taken up by the Kalpasar department on the design flood (i.e., design discharge), location, and configuration, length and discharge capacity of the flood regulator.

15.1.1 Location of the Flood Regulator

The preliminary design proposed by Kalpasar department, is based on a provisional maximum outflow of 100,000 m^3/second the flood regulator. **Figure 15.1** depicts the view of flood regulator location on Google maps. Two alternative locations were initially proposed, the alternative I layout seeks to be placed nearest possible to the coast line, on shallow grounds. This would result in shorter lengths of the approach channel and tail channel. As this alternative is located near the coast line, the impact of waves will be more and hence wave protection structures should be constructed, which in turn adds extra cost to the project. Alternative II layout will be further towards the east, where ground levels are relatively higher and flatter. This will be on salt pans where adequate land would be available for construction plant and machinery. The advantage is that the wave impact will be minimal. But this will require larger lengths of the approach channel and tail channel, which would increase the dredging cost.

Considering the merits and demerits of the two locations, a third alternative is proposed between these two alternatives as shown in **Figure 15.2**. This alternative will start at 4m MSL (Mean Sea Level) which will have lesser wave impact than the alternative I. Also, the smaller approach channel length reduces the dredging quantity which reduces the dredging cost.

15.1.2 Design Length of Flood Regulator

The Department of Hydrology, IIT Roorkee estimated the Design flood PMF and Capacity of Flood Regulator for the Kalpasar Project. The Kalpasar reservoir catchment area has3 major river systems in the north and east namely Sabarmati, Mahi, Dhadhar along with the Narmada diversion canal and the 7 small rivers in Saurashtra to the west, But it is assumed that water from the Narmada River will not be diverted to Bhadbhut Barrage during PMF

event. All of these rivers have controlled structures like dams and barrages etc. for storage and diversion of available water, before these rivers meet the Gulf of Khambhat. PMF for Kalpasar Reservoir was estimated through steps mentioned below.
(a) Estimation of Maximum Rainfall in Catchment Area;
(b) Flood generated by each River Basin; and
(c) PMF at Reservoir by combining Flood from various Basins.

The PMF flood volumes in case of storm centering over Dhadhar, Mahi and Sabarmati have been taken for computations of the design length of the flood regulator. The length of flood regulator is estimated by solving the continuity equation. The inflow in the equation is due to PMF and outflow from the flood regulator is controlled by tide levels. The routing studies were carried out for a PMF hydrograph of 200 hrs, for different lengths of flood regulator and maximum water levels were computed. The tidal levels were selected in such a way that at the start of the PMF the tidal level is also above 3.0 m and the tide is rising. The raise in a reservoir level is limited to El +5.0m. A Flood regulator having 95 spans of 18 m width and 94 no. of 4 m thick piers with a capacity of about 110,000 m^3/s, is recommended. In addition to these 95 spans, 5 more spans are provided (3 spans for sea level rise and 2 spans for maintenance purposes), thus totalling 100 spans of 18m wide accordingly the net width and gross width correspond to 1800m and 2196m respectively.

15.1.3 Crest Level and Configuration of Flood Regulator

A study was taken up to explore the possibility of having crest at higher elevations, which results in larger widths of flood regulator but affords advantage in respect of gate size and longer duration of operation with respect to tides rising above crest levels.

Figure 15.1: Location of Flood Regulator on Google Maps

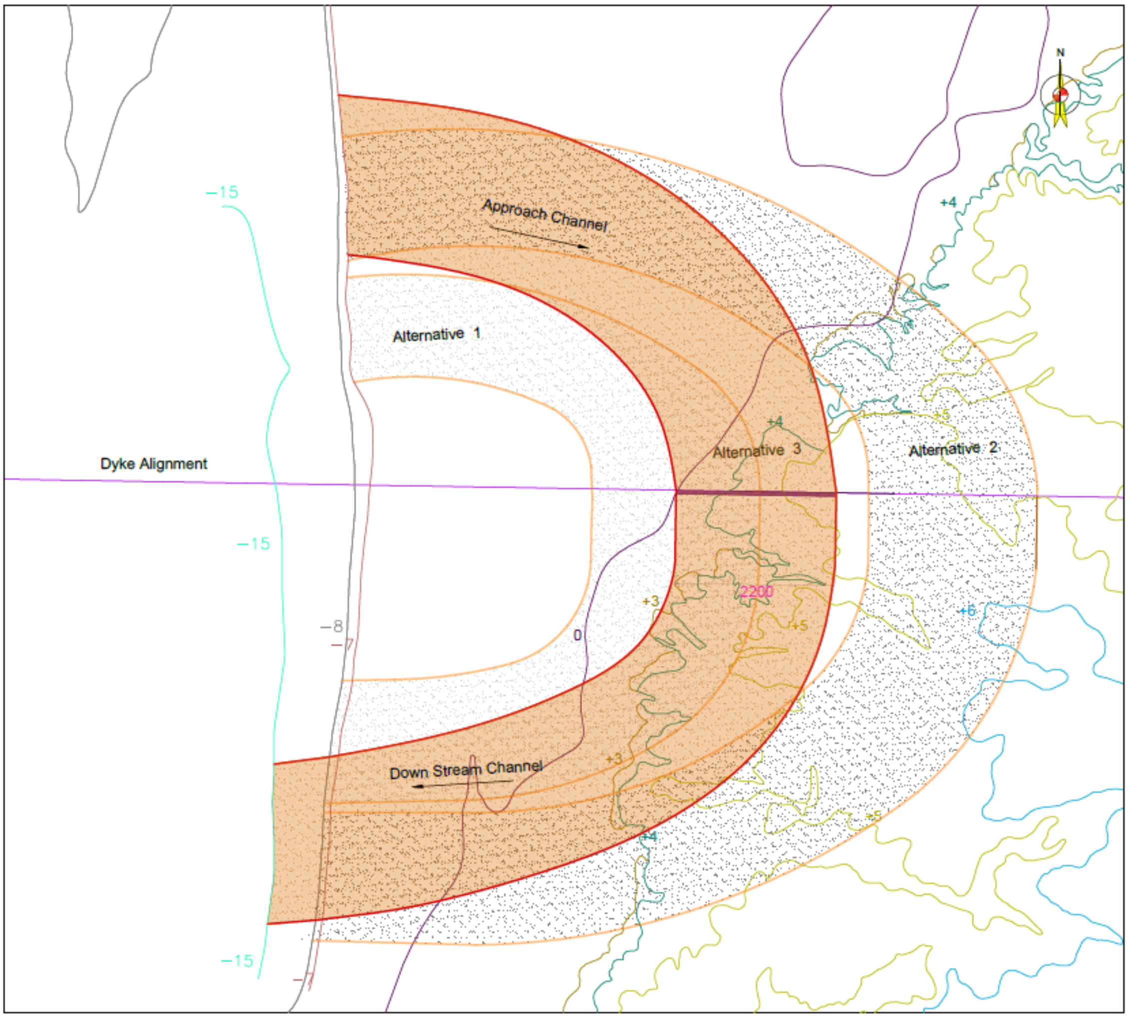

Figure 15.2: General Layout - Location proposal of Flood Regulator (Plan)

Alternative crest elevations of -3.5m, -2m, 0m and +3m were examined from above considerations. Study of the above reveals that it would be impractical to have the crest of the flood regulator higher than El-3.5m or would result in very long flood regulator. It was, therefore, decided to keep the crest level at El -3.5m.

The arrangement proposed for flood regulator section has been designed as a standard WES elliptical profile for a design head of 6.5m corresponding to FRL (Full Reservoir Level), the crest level being at El -3.5m. The maximum operating head corresponding to HFL (High Flood Level) would thus be 8.5m. The span width would be 18m, separated by 4m thick piers and the maximum flood regulator outflow of 1,10,000 m^3/s would require 100 spans, each controlled by vertical lift gates. The channel on upstream side is fixed to El -7.0m to maintain uniform flow of water on the reservoir side before getting discharged. The level of stilling basin on the downstream side is maintained at El -10.0m.

15.2 Components

The plan and longitudinal sections of the entire flood regulator and its components are presented in **Figure 15.3, Figure 15.4 & Figure 15.5.** The components that are designed as a part of flood regulator include:

(a) Waterway;

(b) Ogee Weir;
(c) Energy Dissipation Arrangement;
(d) Scour Protection & Sheet Piles;
(e) Approach Channel;
(f) Spill Channel;
(g) Piers supporting transportation system; and
(h) Retention structure.

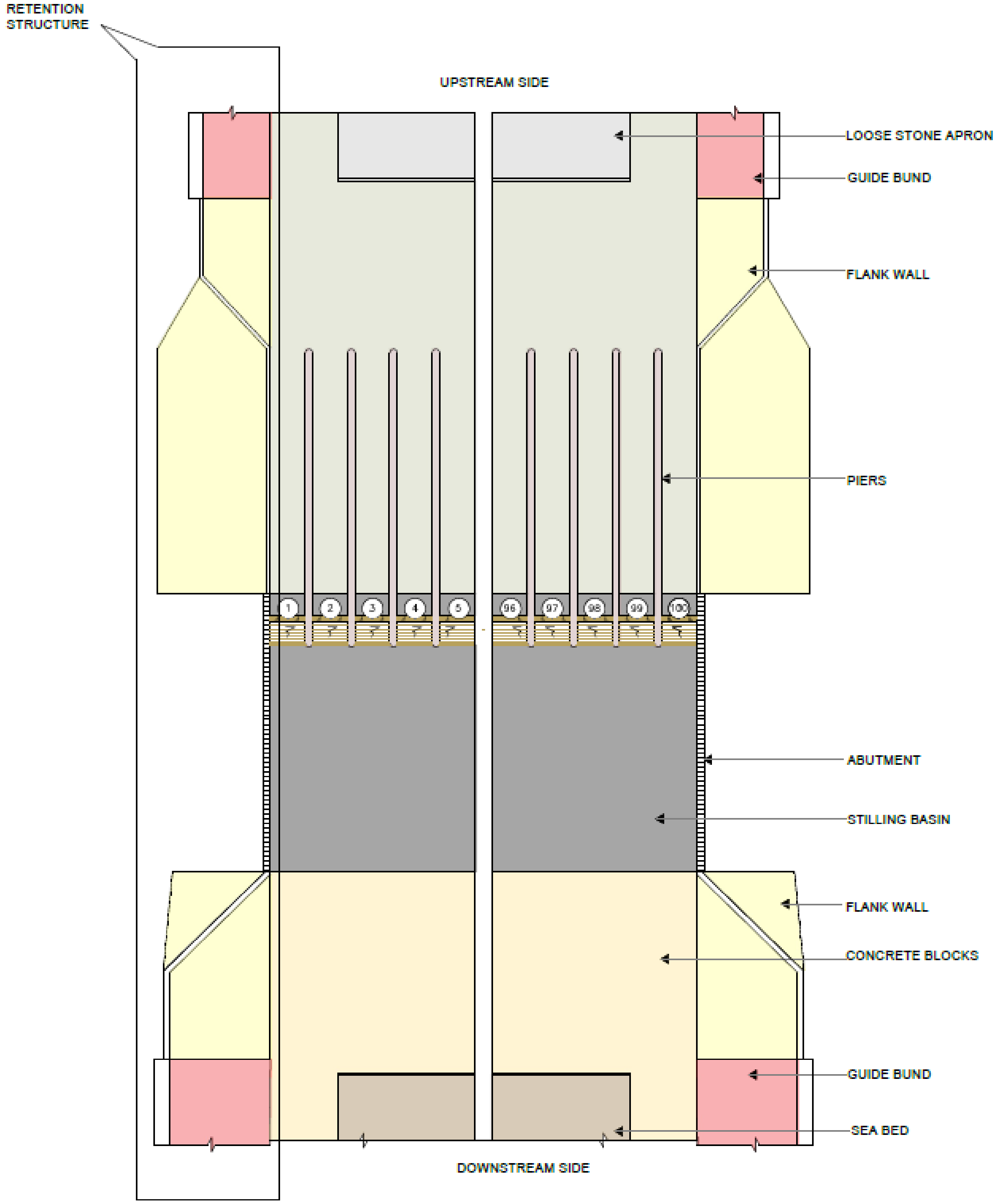

Figure 15.3: Plan of Flood Regulator

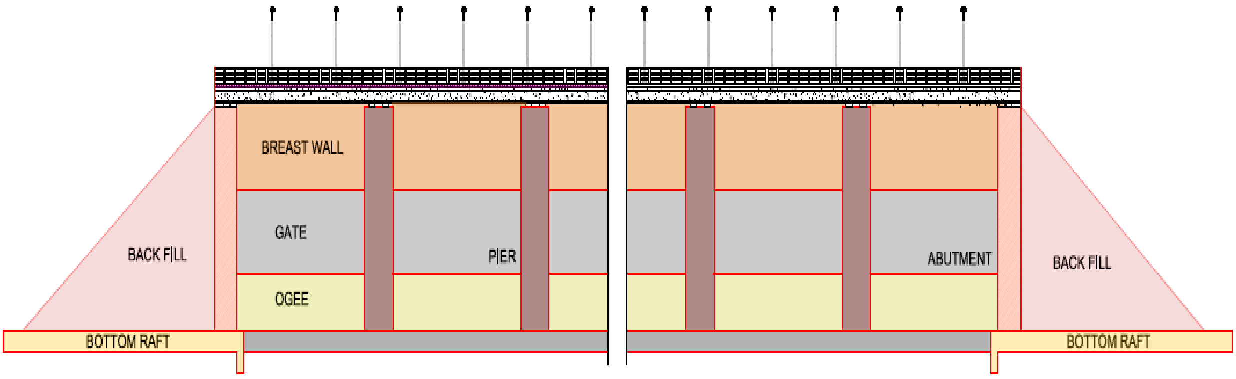

Figure 15.4: Longitudinal View of Flood Regulator at Ogee Section

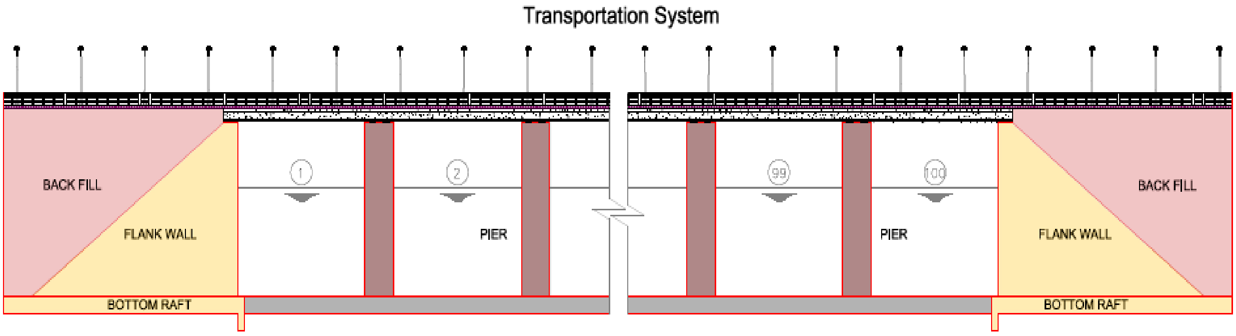

Figure 15.5: Longitudinal View of Flood Regulator at Flank Wall Section

15.3 Waterway

The waterway is dependent upon the design flood. The PMF flood has been adopted as a design flood for designing various components of the flood regulator. The ground level of the reservoir bank has been limited to El (+) 5.0m to avoid excess inundation at dyke and land site.

The details of the proposed waterway are as under:
(The levels mentioned below are Elevation levels (El))

Lowest Bed Level on Upstream	=	(-) 7.0	m
Lowest Bed Level on Downstream	=	(-) 12.0	m
Crest Level of flood regulator	=	(-) 3.5	m
Full Reservoir level (FRL)	=	(+) 3.0	m
High Flood level (HFL)	=	(+) 5.0	m
Clear Span between Pier to Pier/Abutment	=	18	m
Thickness of Pier	=	4	m
C/C of Piers	=	22	m
No. of flood regulator bays	=	100	bays
Type of Gate	=	Vertical Lift Gate	
No. of gates	=	100	gates
Top level of flood regulator piers on Upstream	=	(+) 6.5	m

Top level of Upstream abutments	=	(+) 6.5	m
Top level of Approach channel	=	(+) 3.0**	m
Top level of Upstream flank wall	=	(+) 6.5	m
Top level of flood regulator piers on Downstream	=	(+) 14.5	m
Top level of Downstream abutments	=	(+) 14.5	m
Top level of Spill channel	=	(+) 3.0**	m
Top level of Downstream flank wall	=	(+) 9.0	m
Total water way width (incl. Piers)	=	2232	m
Effective waterway, Le	=	1761	m

**

Originally it is proposed by NCCR that top levels of approach and spill channels are +6.5 m and +9 m respectively. The sandfill embankment on the approach and spill channels from EL + 3m to +6.5 m and + 9 m will not be feasible, as dredging and filling up of material for making channel slopes in sea is not practical. Therefore, the sandfill embankment on the top of channels is not recommended. The approach and spill channels will be formed by taking natural slopes.

The length of the waterway that is required in a flood regulator to discharge the water from upstream to downstream is estimated. The waterway can be estimated using design discharge Eq (15.1)

$$Q = \frac{2}{3}\, C.\sqrt{2g} L_e {H_e}^{1.5} \quad (15.1)$$

Where,
C is the coefficient of discharge;
L_e is the clear waterway to discharge the water from the reservoir into sea; and
H_e is the effective head which includes the design head and approach velocity head.

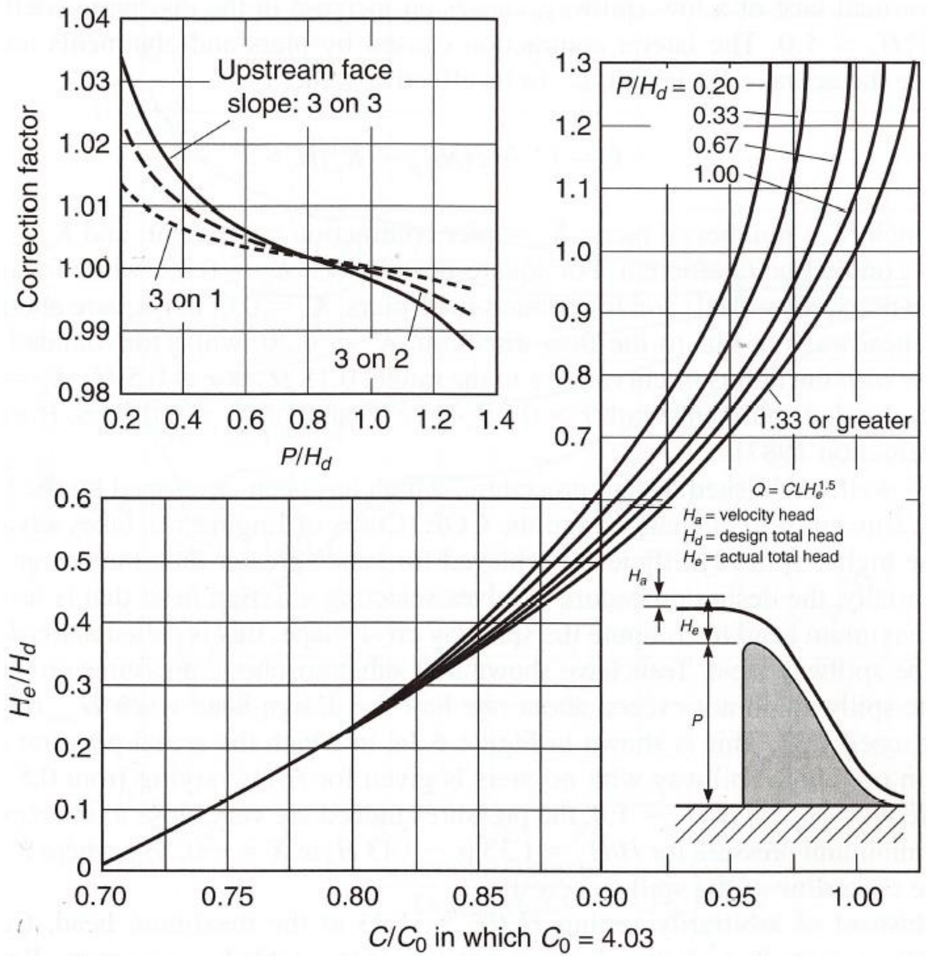

Figure 15.6: Coefficient of Discharge for an Ogee Weir with sloping Upstream Face

Ogee weir shape is adopted as suggested by Waterways Experiment Station (WES). 1:1 slope in the upstream face is adopted. Based on the P (Crest Height)/H_e ratio, C is obtained from **Figure 15.6**, which is valid for upstream vertical face. The coefficient thus obtained is increased by a factor based on the slope of the upstream face. The factor depends upon the upstream slope and P/H_d value. This is obtained from the book Open Channel Hydraulics by Ven te Chow.

For the present conditions of P = 3.5 m; H_e = 9.4 m; C = 0.738 and Q = 110,000 m^3/s, the required$L_e$is1761 m. The total waterway width is 2232 m. With 100 bays and a pier width of 4.0 m, L_e provided will be1800 m. The additional width is provided to accommodate for Sea Level Rise (SLR).

15.4 Hydraulic Design

The general arrangement of the main components of the flood regulator, along with the spill channel is shown in **Figure 15.7**.

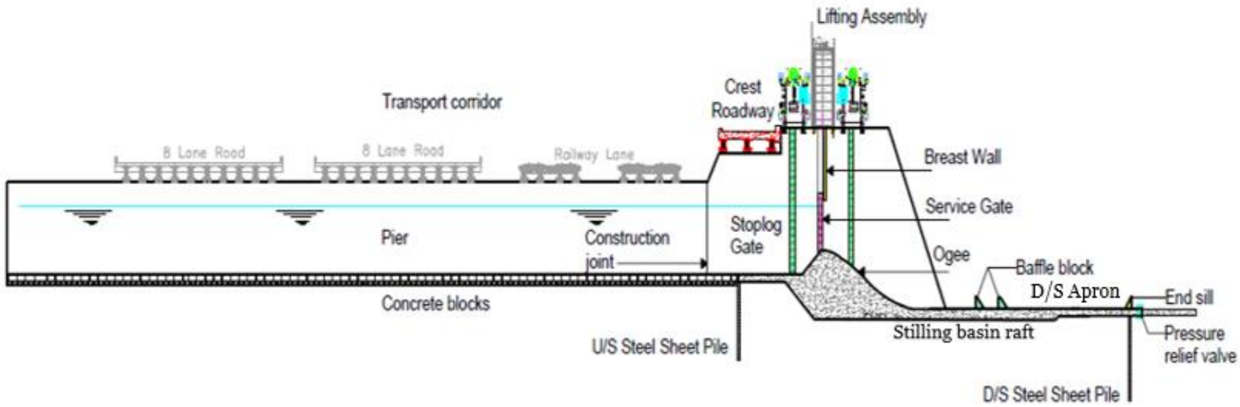

Figure 15.7: Components of Flood Regulator

15.4.1 Ogee Weir

The IS code for flood regulators IS 6934 (1998): Recommendations for hydraulic design of high ogee overflow spillways [WRD 9: Dams and Spillways] is valid for high head flood regulators. The IS code for barrages IS 6966-1 (1989): Hydraulic design of barrages and weirs - Guidelines, Part 1: Alluvial Reaches [WRD 22: River Training and Diversion Works] does not contain any information on ogee weirs. Waterways Experiment Station (WES) flood regulator shape with 1:1 upstream face is adopted. Details of the WES flood regulator is provided in the book Open Channel Hydraulics by Ven te Chow. The shape of the flood regulator face is shown in **Figure 15.8** below.

Eq (15.2) gives the profile of the weir on the downstream face i.e., to the right of origin O. This curve joins smoothly to the cistern bed level at -12.0 m. That is maximum value of y will be 8.5 m. The radius of the upstream curve, R = 3.825 m.

$X^{1.776} = 9.857 \times Y$(15.2)

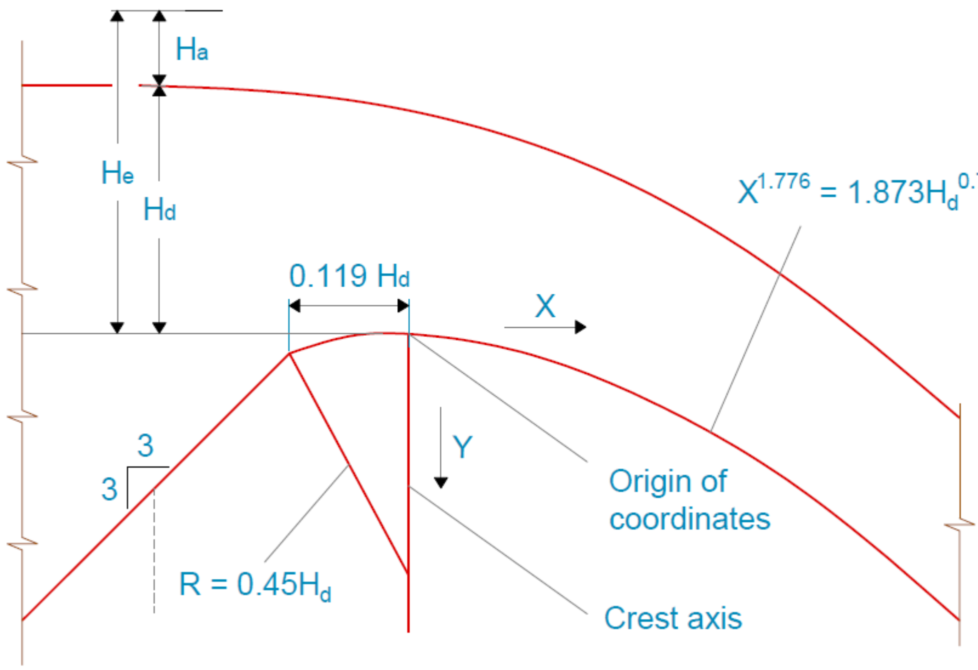

Figure 15.8: Shape of Ogee Weir

15.4.2 Energy Dissipation Arrangement

As per IS standard, the level of stilling basin may have to be set as low as El (-)24m when full discharge of 110,000 m3/s is considered and low tide level on the downstream side. But this is not feasible for construction. Therefore, the downstream floor level of the stilling basin is proposed at a higher elevation. This was suggested even in the earlier design proposed by Kalpasar department. It was suggested that the stilling basin level may be set at El (-)10.0 m. But by keeping the floor at higher level than at the required level, the energy may not be dissipated within the stilling basin and the hydraulic jump will sweep downstream. Hence there will be detrimental effects on the downstream side by way of erosion of the bed due to very high velocities. But, considering the constraints of dredging cost, level of stilling basin is set at a higher elevation, and designing the stilling basin, as per the recommendations provided by U.S. Army Corps of Engineers (USACE). To stabilize the jump within the stilling basin, spill channel is designed accordingly. The schematic of stilling basin, as suggested by the USACE is shown in **Figure15.9**.

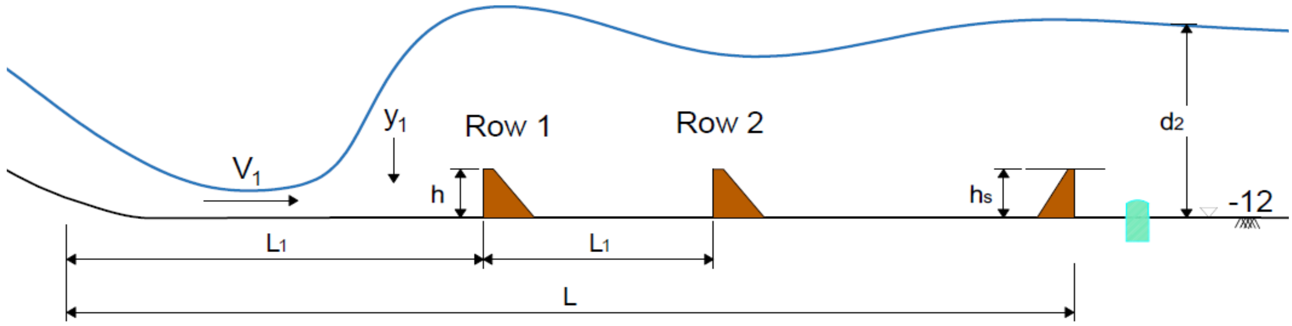

Figure 15.9: Schematic diagram of the Stilling Basin and Energy Dissipation Arrangement

The floor level of the stilling basin is set at El (-)12.0 m. The stilling basin is provided with two rows of baffle blocks and one end sill. Various dimensions in the **Figure 15.9** are as follows:

L_1	=	19.0 m;
h	=	2.1 m;
L_2	=	5.3 m;
h_s	=	1.1 m;
L	=	50.0 m;
Width of baffle block	=	2.0 m;
Spacing between the baffle blocks	=	2.0 m; and

Baffle blocks in rows 1 and 2 are staggered.

For the case of a discharge of 110,000 m3/s, the hydraulic jump will form in the stilling basin as long as the depth downstream of the jump is 12.6 m. It is also estimated that the hydraulic jump will form in the stilling basin even for lower discharges of 60,000 m3/s and low tide levels on the downstream side.

15.4.3 Scour Protection

There is a concrete floor on which ogee weir and the energy dissipater will be placed as shown in **Figure 15.10**. Sheet piles are provided on the upstream side and downstream side of the concrete floor to prevent the scour holes from propagating underneath the floor.

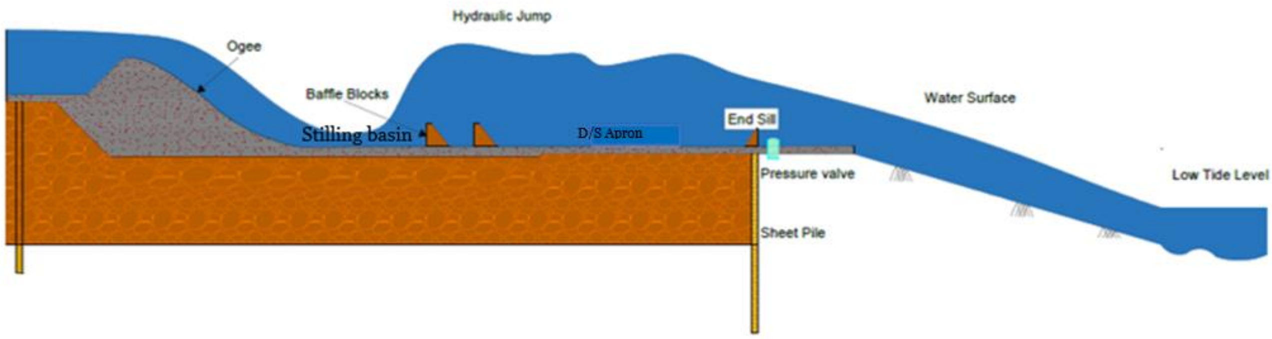

Figure 15.10: Schematic diagram showing Concrete Floor with Sheet Piles

The sheet piles are designed based on the scour depth. The scour depth, as per Lacey's theory is 23.2 m, for a material size of 0.34 mm. As per IS 6966-1 (1989): Hydraulic Design of Barrages and Weirs - Guidelines, Part 1: Alluvial Reaches [WRD 22: River Training and Diversion Works], a factor of safety is used for finding the size of sheet piles, in case of alluvial channels. In the present case, channels are lined with concrete on both sides of the flood regulator. Therefore, the depth to which sheet piles need to be taken is based on the scour depth R as such, without any FoS.

(a) Upstream Sheet Pile

Bottom level = +5 -23.2 = -18.2 m.
Size of Upstream Cutoff = 18.2 -7.0 = 11.2 m
Suggested size of the Upstream Sheet Pile for cutoff = 11.5 m

(b) Downstream Sheet Pile

For a flow of 110,000 m^3/s, the expected water level on the downstream side = -1.3 m
Bottom level = -1.3 -23.2 = -24.5
Size of Downstream Cutoff = 24.5 -12.0 = 12.5 m
Suggested size of Downstream Sheet Pile for cutoff = 12.5 m

The flow rate through the flood regulator is highly variable. Similarly, the tidal levels vary from -5.3 m to +6.2 m. These variations introduce uncertainty regarding the formation of the jump within the 50 m long stilling basin. Therefore, as an additional factor of safety, an extra concrete floor thickness of 2.0 m should be provided beyond the stilling basin for a length of 50 m. On the downstream side of the stilling basin, a row of pressure relief valves should be provided for relieving the uplift pressures at this location.

There is no need for providing other protection measures that are usually provided in case of flood regulators in alluvial channels, such as lined concrete channels on both upstream and downstream sides.

15.4.4 Uplift Pressures

The total length of the concrete floor is determined based on the exit gradient. We have used a factor of safety of 7.0 while calculating the critical exit gradient. Provide a Floor Length of 76 m. The Floor Length required on the upstream side is 26m. The Minimum Floor Length required on the downstream side is 50m. The Secondary Floor Length required on the downstream side is 50m.

Uplift pressures are calculated based on Khosla's theory. Two critical cases are considered: (i) Upstream water level at FRL (pond level), no flow and downstream water level at corresponding to LTL and (ii) Upstream water level at HFL, Q = 110,000 m^3/s

occurring with hydraulic jump in the stilling basin and Downstream water level at -1.3 m corresponding to the PMF. Net unbalanced uplift pressure heads are calculated and then the thickness of the concrete floor is calculated.

Net unbalanced pressure upstream of the ogee is negative. Therefore, nominal concrete thickness of 1.0 m is provided.

Net unbalanced pressure head at the toe of the ogee weir = 10.5 m (occurs for the critical case (ii)). Therefore, the thickness of concrete required at this location is7.5 m.

Net unbalanced pressure head at a distance of 15.0 m from toe = 7.83 m (occurs for the critical case (ii)). Therefore, the thickness of concrete required at this location is5.6 m.

Net unbalanced pressure head at a distance of 35 m from the toe is 4.5 m. therefore; the thickness of concrete required at this location is 3.2 m.

Earlier, we proposed raft thickness of stilling basin varying from 2.5m at the beginning, followed by 1.5m and 0.8m along the length. Subsequently after the virtual meeting held on 7th November 2022 with Prof.B.S. Murty, the raft thickness has been modified to 7.5m at the beginning and followed by 5.2 m and 3.6 m along the length to counter act the uplift pressure. The revised proposal is to adopt a uniform raft thickness of 2.5 m along the length of the stilling basin. For this raft thickness, the cellular cofferdam design has been done and is given in Enclosure 3. According to this design, the cellular cofferdam will require sheet piles of length varying from 56 m to 70 m depending on the strata conditions (example for BH C1, length required is 66 m; for C2, it is 66.5 m; for C3, it is 56 m; and C7, it is 70 m). The design has also been carried out for 7.5m raft and the length of the sheet pile required for cofferdam is varying from 69 m to 87 m. It is impractical to construct cellular cofferdams for 7.5m thick raft. Even for 2.5m thick raft, it will be a challenge. But there is no other option.
Hence, it is recommended to adopt 2.5m thick raft. To counteract the uplift pressure on raft, bored cast in-situ piles are proposed. The detailed design is given in Enclosure 2.

15.4.5 Approach Channel

An approach channel is of trapezoidal cross-section. It should be 2200m wide at the bottom and should have 1H: 1V side slopes. For a length of 2.0 km from the flood regulator, the channel should be lined with concrete. Beyond this point, the reservoir should be dredged to create a deep pool from where water can enter into the channel easily. Pool of water it should be simply dredged in the reservoir. The longitudinal slope of channel $S_o$is 0.00012.

15.4.6 Spill Channel

The construction of an appropriate spill channel is essential for maintaining required water depth in the stilling basin, which facilitates the formation of the hydraulic jump. The spill channel should be of trapezoidal shape, with a bed width of 2200 m and side slopes of 1H: 1V.

The longitudinal slope of the spill channel is 0.00025. It should have a Manning roughness coefficient value greater than 0.0175. The total length of the spill channel should be 10.0 km. The bed elevation of the channel at the downstream end is -14.5 m. Beyond this point, the estuary should be dredged to have a deep receiving pool.

15.4.7 Recommendations

Following are the important recommendations proposed,

(i) Passing a flood of 110,000 m^3/s (PMF) over the flood regulator, with an HFL of +5.0 will increase the water level to +6.1 m in the reservoir. Thus, the inundation will occur up to a contour level of +6.1 m. If the inundation level is to be kept at +5.0 m, additional 26 bays have to be added to the flood regulator. This will not only increase the cost by almost 30%, it will also create significant operational difficulties. There will not be inundation beyond +5.0 m contour, as long as the flood discharge is less than 87,000 m^3/s. As the PMF occurs rarely, it is not prudent to increase the length of the flood regulator. On the contrary, other flood protection works are recommended for protecting strategic infrastructure that may be located in the vicinity of the reservoir.

(ii) Flood regulator is recommended to have a total of 100 bays, with each bay having a width of 18 m. The total length of the flood regulator is 2200 m. The bed level upstream of the flood regulator is proposed at El -7.0 m.

(iii) A 2200 m wide approach channel, with a slope of S_0 = 0.00012 is recommended on the upstream side and it should be lined with concrete for a length of 2000 m from the flood regulator.

(iv) The crest level of the ogee weir is proposed at El -3.5 m. The ogee weir is a low head ogee weir. The shape of the ogee weir corresponds to the standard shape suggested by the Waterways Experiment Station, with a 1:1 upstream face.

(v) The hydraulic design of the flood regulator recommends the stilling basin at a level of El -12.0 m. The concrete floor of the stilling basin should extend for a length of 26 m from the toe of the ogee weir on the upstream side. This part of the concrete floor should have a nominal thickness of 1.0 m. The concrete floor thickness should be 2.5 m for the entire length of the stilling basin (Downstream side).

(vi) Energy dissipation mechanism in the form of baffle blocks and end sill are recommended to be proposed on stilling basin. A row of baffle blocks of height 2.1 m; width 2.0 m and spacing of 2.0 m should be provided at a distance of 19 m from the toe of the ogee weir. Another row of same size baffle blocks (but staggered in position) should be provided at a distance of 5.3 m from the first row. An end sill (continuous) of height 1.1 m should be provided at the downstream end of the stilling basin.

(vii) An additional length of the concrete floor of thickness 2.0 m is recommended for a length of 50 m considering the safety aspects.

(viii) It is recommended to provide a steel sheet pile of size (depth below concrete floor should) 11.5 m on the upstream side and a steel sheet pile of size (depth below concrete floor should) 12.5 m on the downstream side.

(ix) A 10.0 km long spill channel, with a slope of 0.00025 is recommended downstream side of the flood regulator. The width of the spill channel should be 2200 m. It should have a Manning roughness coefficient more than 0.0175.

15.5 Geotechnical Design

The design is based on the geotechnical investigation carried along the flood regulator location. A total of 35 investigations combining 22 boreholes and 13 ECPT (Electro-Cone Penetration Test) were carried out in the flood regulator region apart from the previously available data. The critical profiles along the components are identified. The components and the corresponding boreholes considered are presented in **Table 15.1** and the borehole locations are shown in **Figure 15.11.**

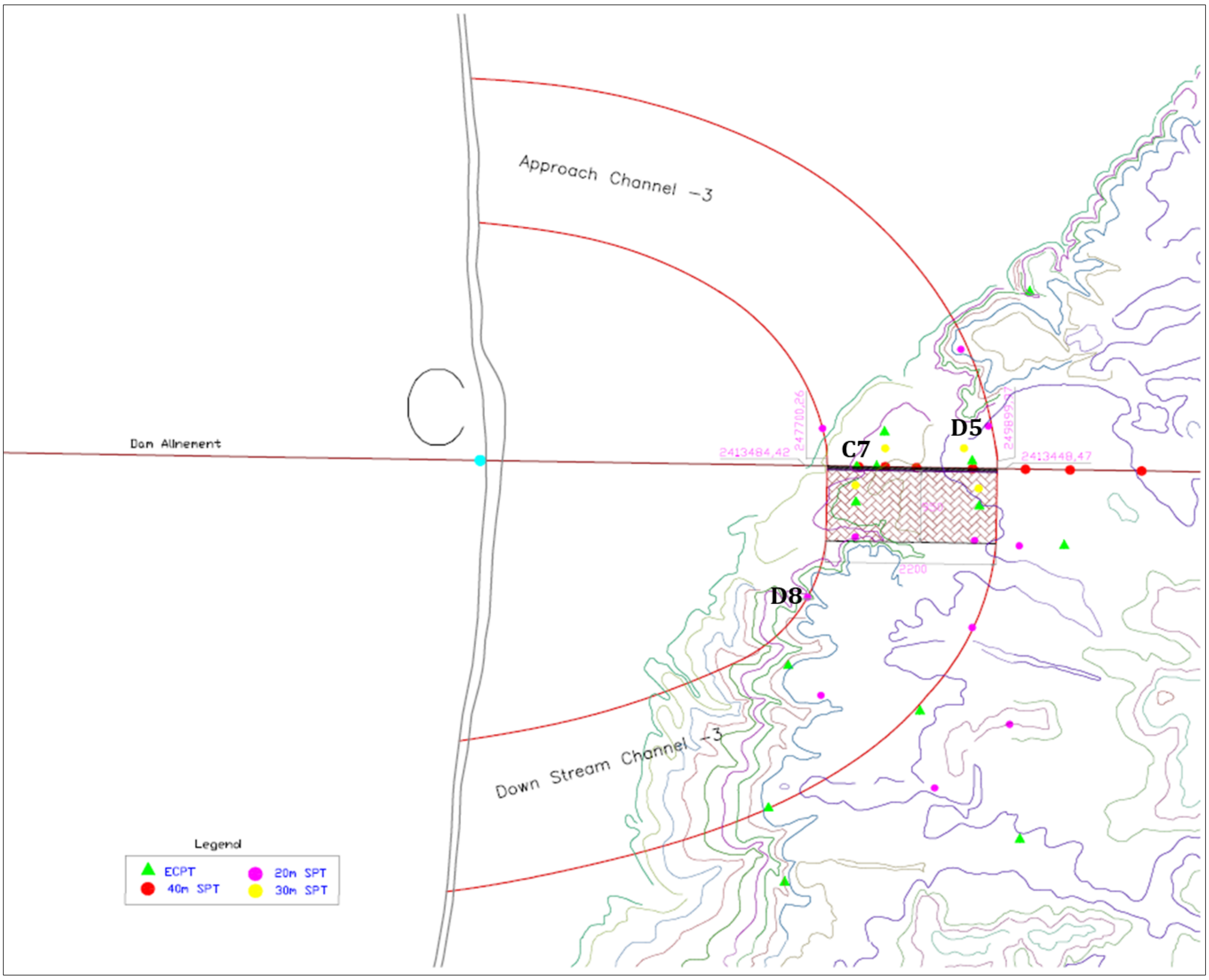

Figure 15.11: Boreholes and the critical boreholes adopted for Design

Table 15.1: Geotechnical Data

Field Tests	Flood Regulator	Approach channel	Spill channel
Boreholes	C1, C2, C3, C7	A2, A4, D5, D6, D7	A1, A3, D1, D2, D4, D8, D11

15.5.1 Foundation for Flood Regulator

The foundation of the flood regulator provides the details of the thickness of the raft under the flood regulator, the bearing capacity details and the settlement analysis under the raft.

(a) Thickness

The thickness of the stilling basin has arrived in reference to the uplift calculations given in section 15.4.4. The schematic drawing showing the varying thickness of stilling basin is given below in **Figure 15.12.**

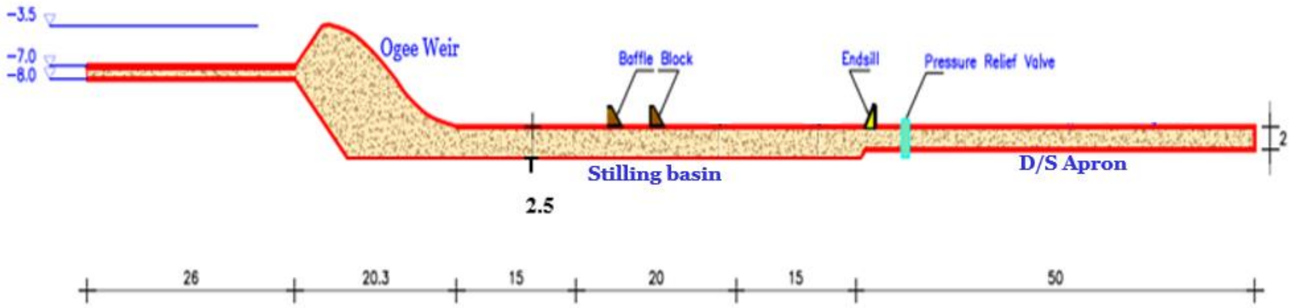

Figure 15.12: Schematic cross section of Stilling Basin Thickness

(b) Bearing Capacity

The bearing capacity of the soil should withstand the loads imposed on the soil. Due to the varying thickness of the raft as shown in **Figure 15.13** in the ogee and stilling basin, the safe bearing capacity is calculated piecewise. The bearing capacity is calculated based on IS 6403: 1981 for shallow foundations on the soil. The local shear failure is considered for SPT values less than 10 and general shear failure is considered for SPT values greater than 30for bearing capacity calculations. The bearing capacity for each section is tabulated in **Table 15.2**.

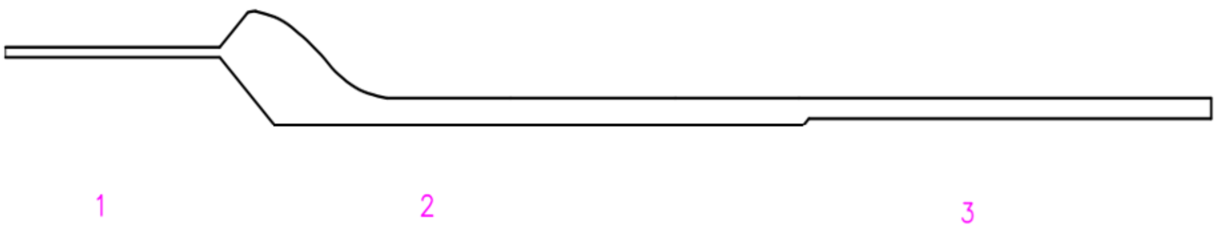

Figure15.13: Different Sections considered for SBC

Table 15.2: Safe Bearing Capacity

Section	Raft thickness (m)	Safe Bearing Capacity (SBC) t/m²
1	1	90
2	2.5	>300
3	2	>300
U/S Flank Wall	2	50
D/S Flank Wall	3	280

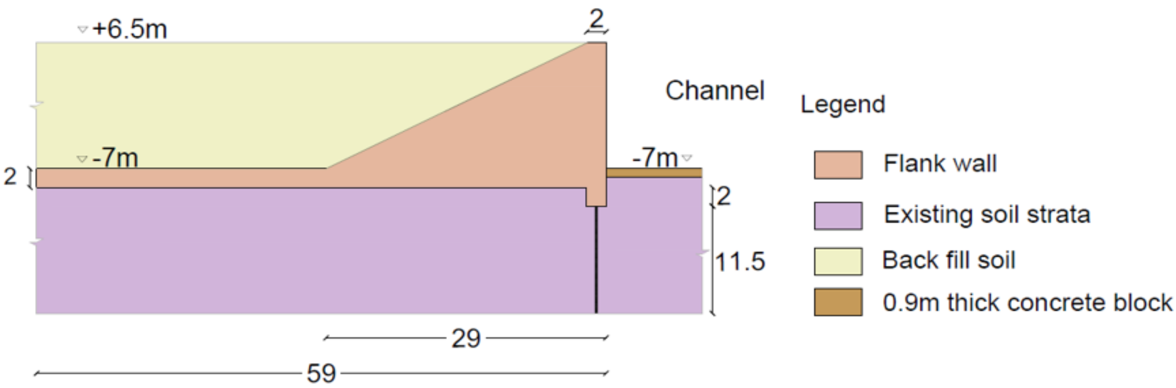

Figure15.14: Cross Section of Upstream Vertical Flank Wall

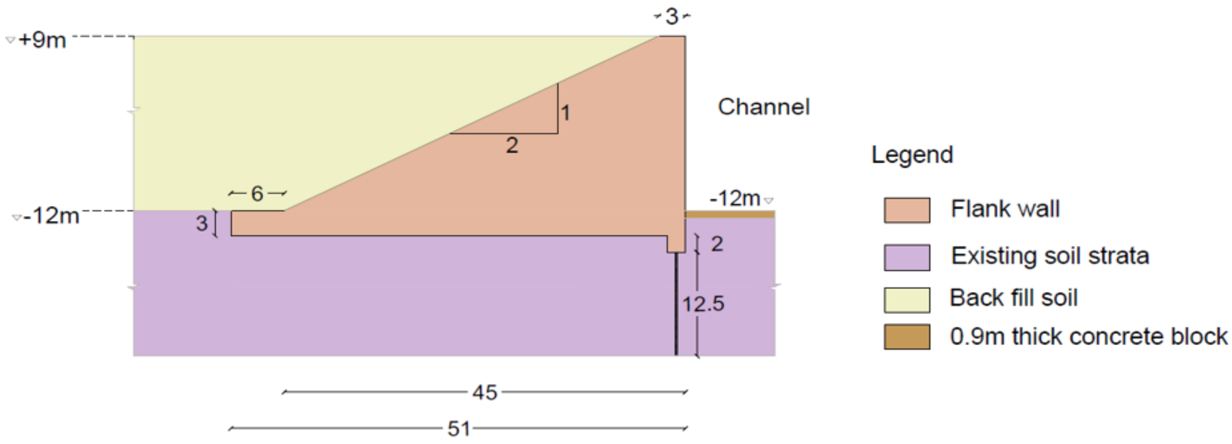

Figure15.15: Schematic cross section of Downstream Vertical Flank Wall

(c) Settlement analysis

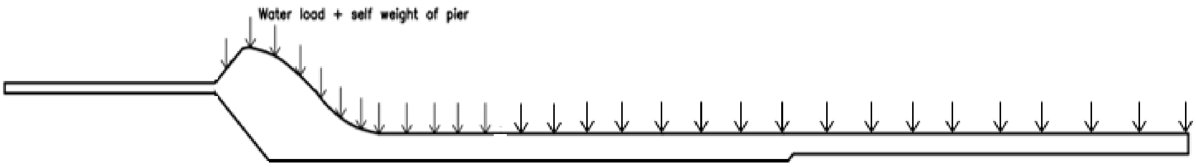

Figure 15.16: Typical Section of the stilling basin raft

The maximum settlement obtained is 65 mm, which is within the permissible limits.

(d) Pile Foundation under stilling basin

Pile foundation is recommended to support the stilling basin to counteract the uplift forces. The safe axial, uplift and lateral pile capacities (free and fixed head conditions) capacities for bored cast in-situ (BCIS) piles have been estimated as per IS: 2911 (Part 1 / Sec 2) – 2010.
Recommended pile diameters and length are 1 m / 1.5 m and 20 m respectively. **Figure 15.17** shows the typical cross section of pile foundation under stilling basin.

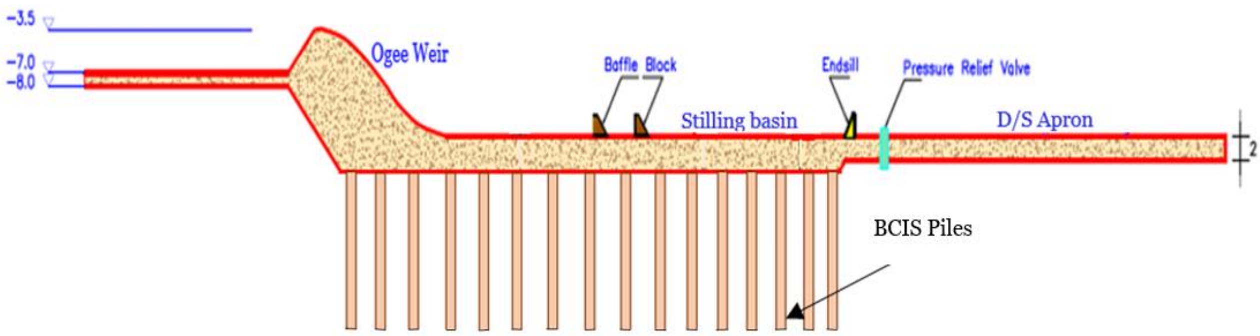

Figure 15.17: Schematic diagram showing pile foundation under stilling basin

The number of piles and spacing are to be calculated depending on the choice of diameter of pile from ease of execution. In our view, both 1 m and 1.5 m diameters are technically equal.

The range of safe axial and uplift capacities are 370 t to 425 t & 160 t to 180 t respectively for 1 m diameter of pile. Similarly for 1.5 m diameter of piles, the range of safe axial and uplift capacities are 925 t to 985 t & 225 t to 290 t respectively. **Table 15.**3 and **Table15.4** represents the number of piles and spacing for various diameters corresponding to different boreholes.

Table 15.3: Number and spacing required for 1 m dia. and 20 m length of piles

BH	Total number of piles required	Spacing between piles (m)
C1	8340	4.2
C2	11015	3.7
C3	9500	3.9
C4	8170	4.25

Table 15.4: Number and spacing required for 1.5 m dia. and 20 m length of piles

BH	Total number of piles required	Spacing between piles (m)
C1	5225	5.3
C2	6785	4.7
C3	5890	5
C4	5280	5.3

15.5.2 Stability Analysis

(a) Abutment and Wing Wall

The abutment forms the end piers of the floor regulator. The abutment is integrated with the raft at the bottom. The abutment will therefore be stable. The earth pressure is estimated for structural design purposes. The earth pressure forces on the abutment are similar to that of the wing wall and can therefore be used for the wing wall as well.

(b) Flank Wall

The flank wall acts as a transition between the vertical walls of the abutment and the inclined facing of the guide bund. Two critical sections of the flank wall are analyzed for stability: the section just adjacent to the abutment and that to the guide bund. Flank walls on the upstream side is proposed as shown in **Figure 15.14** and on the downstream, it is proposed as shown in **Figure 15.15**. Both static and pseudo-static cases have been analyzed to meet the safety criteria. The flank wall is analyzed for the construction and post-construction phases. The surcharge during construction and post-construction is assumed to be constant. The summary of the design is provided below.

(i) Upstream Side near Abutment

This acts as a transition between the abutment's vertical wall and the slope of the guide bund. The transportation corridor rests on the top of the flank wall and therefore a surcharge of 100kPa is utilized in the design as per IRC 6: 2014. The water level during construction and post construction is given in **Table 15.6.**

(ii) Downstream Flank Wall near Abutment

This acts as a transition between the abutment's vertical wall and the slope of the guide bund on the downstream side. A surcharge of 30kPa is utilized in the design as per IRC 6:2014 towards the vehicular loads that might access the location during and post-construction. The water level during construction and post construction is given in **Table 15.7**

Table 15.6: Water Levels in different Project Phases on Upstream side near Abutment

Phase	Highest Level	Lowest Level
Construction Phase	+5.2 m	-4.8 m
Post-construction Phase	+5 m	-4 m

Table 15.7: Water Levels in different Project Phases on Downstream side near Abutment

Phase	Highest Level	Lowest Level
Construction Phase	+5.2 m	-4.8 m
Post-construction Phase	+8.1 m	-5.3 m

(c) Approach channel

The current ground level is EL + 3 m. The channel bed level is EL -7 m. As per the latest soil investigation data, the strata from EL +3 m to EL -5 m (8 m depth) is very soft silty clay with SPT N of 0. This material will behave like fluid and will get eroded with the flow. The recommended slope for the channel from EL -5 m to EL – 7 m is 1V:1H. The factor of safety obtained is greater than the required FoS as per IS.

(d) Spill channel

The current ground level is EL + 3 m. The channel bed level is EL -14.5 m. As per the latest soil investigation data, the strata from EL +3 m to EL -6.5 m (9.5 m depth) is very soft silty clay with SPT N of 0. This material will behave like fluid and will get eroded with the flow. The recommended slope for the channel from EL -6.5 m to EL – 14.5 m is 1V:1H. The factor of safety obtained is greater than the required FoS as per IS.
As definite channel is required on the spill channel side, cellular cofferdam is to be adopted. Height of the cellular cofferdam is 38m. The detailed design is given in Enclosure 3.

15.5.3 Channel Protection

The suitable channel and bed protection works for both approach and spill channel are revet mattresses. For velocities ranging from 5 m/s to 6 m/s and a bank slope of 1:2.5, the thickness of the mattress required will be equal to or more than 0.45 m as per IRC: SP:116-2018. On the conservative side, revet mattress of 0.5 m thick is proposed on the bed and slopes of the channels. The maximum wave height that the flood regulator can experience both on the upstream and downstream side is 2m. A maximum thickness of the equivalent gabion armour thickness of 0.5m with the stone diameter, D50, should be at least 0.25m. The size of the stone required will be 0.25 m with a thickness of 0.5m to be stable against flow velocity and hydrodynamic loading from waves.

Since the thickness of revet mattress is more than 0.3 m, two compartments 0.25 m thick are to be provided. The mattress shall be subdivided into 2 compartments by the insertion of a diaphragm made of the same mesh as the rest of the mattress. The diaphragms shall be secured in the proper position at the base with a continuous spiral wire, in such a manner that no additional tying at the junction will be necessary.

The proposed revet mattress consists of mechanically woven double twisted hexagonal shaped wire mesh and a non-woven geo-textile as a filter material. The rock fill in revet mattresses fascia shall be hard, angular to round, durable and of such quality that they shall not disintegrate on exposure to water or weathering during the life of the structure. The stone size shall be ranging between 1.5 to 2.5 times the mesh openings. Each size may allow a variation of 5% oversize or 5% undersize or both. **Figure 15.18** shows a typical revet mattress.

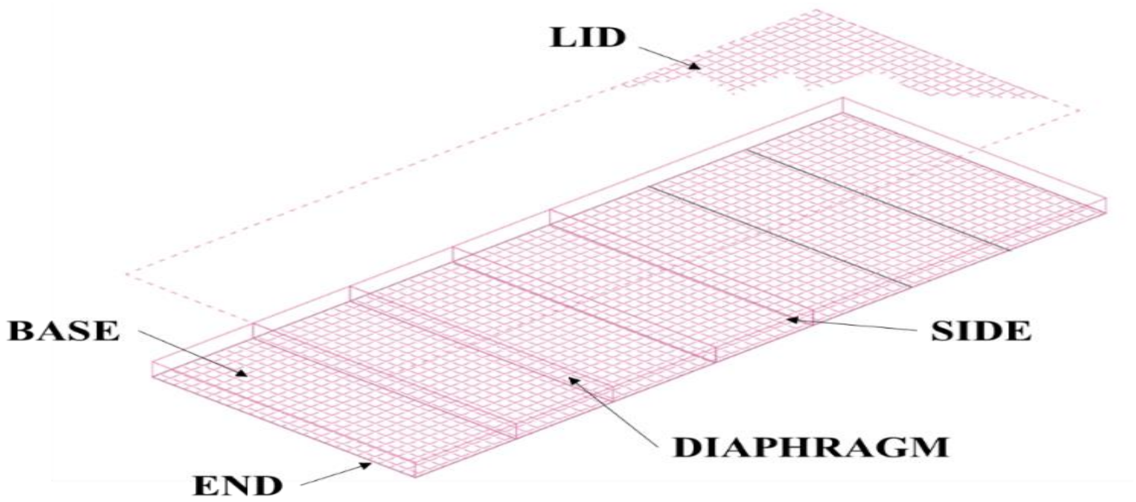

Figure15.18: Schematic diagram showing Revet mattress Configuration

15.5.4 Cofferdam

The flood regulator can be constructed only after the construction of cofferdams on the upstream and downstream sides. Cofferdam restricts the water from entering the construction area. The water present within the cofferdam can be dewatered before construction and the bed can be excavated up to the level of the stilling basin. Earthen cofferdam, rock fill cofferdam, single-walled and double-walled cofferdam, braced cofferdam and cellular cofferdam (circular or diaphragm type) are some of the common cofferdams. Based on the depth of water and soil to be retained, the type of cofferdam to be constructed is selected. Since the depth of soil and water to be retained in this case is more than 20 m, a cellular cofferdam is suitable for this case.

(a) Cellular Cofferdam

A cellular cofferdam comprises interconnected cells that form a water tight wall. These cells are filled with soil to provide stability against various lateral forces.

There are two types of cellular cofferdams such as diaphragm type and circular type. The circular cellular cofferdam is more stable than the diaphragm type and can withstand more lateral pressure due to the high interlocking tension between adjacent cells. **Figure 15.19** shows the plan of circular cellular cofferdams around the flood regulator.

(b) Dimensions of Cofferdam

The cellular cofferdam has been checked against sliding and overturning. Considering the diameter of the cofferdam as 23.95 m, the factor of safety values for various failure mechanisms is greater than the required values. The design of cofferdam calculations and check for stability against various parameters are given in **Table 15.8.**

From IS 9527 part 4, the dimensions of each cell for the corresponding diameter of 19.91 m is given in **Table 15.9.** The plan of cellular cofferdam showing all dimensions is given in **Figure 15.20**.

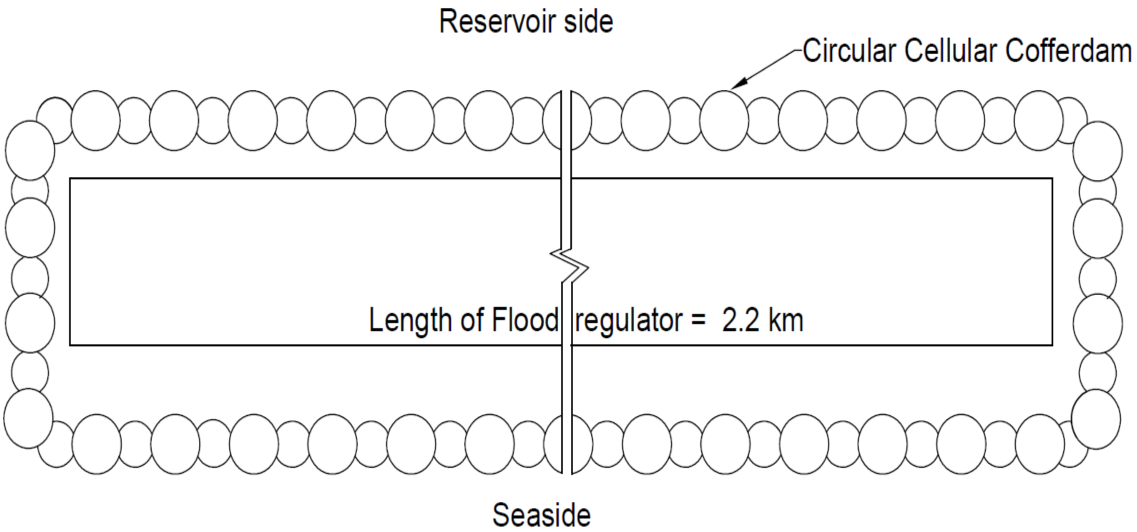

Figure 15.19: Plan showing Cofferdam around Flood Regulator

Table 15.8: Factors of Safety for Various Parameters

Mechanism	Required FoS	Obtained FoS
Sliding	1.25	2.9
Overturning	2	2

Table 15.9: Design Parameters for Cofferdam Design

S. No	Design parameters	Dimensions(m)
1	Effective Width of cell (B)	19.9
2	Radius of Connecting Cells	5.86
3	Edge to edge distance between main cells	1.27
4	Center to center distance between main cells	25.2
5	Number of piles in cell	188

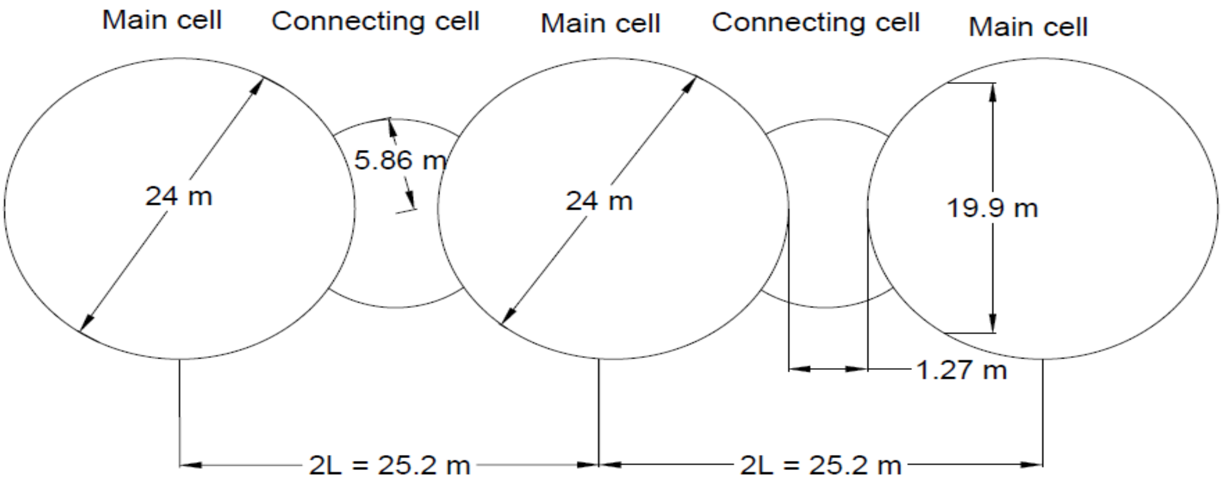

Figure 15.20: Plan of Cellular Cofferdam

15.5.5 Recommendations

Following are the recommendations on the Geotechnical aspects of the Flood Regulator.

(i) From geotechnical aspects retention structures namely Abutment, Flank wall and Guide Bunds are recommended on both upstream and downstream of flood regulator to retain the embankments;

(ii) A slope of 1V:1H is recommended to protect the approach channel bund from EL -5 m to EL – 7 m.

(iii) A slope of 1V:1H is recommended to protect the approach channel bund from EL -6.5 m to EL – 14.5 m.

(iv) Revet mattress has been recommended to protect the channel side slopes against varying hydraulic flow conditions.

15.6 Structural Design

The structural details of flood regulator components namely the breast wall, Ogee weir, Wall piers, Retention structures, Energy Dissipating elements and Protection works are covered as under.

15.6.1 Breast Wall

The top level of the hydro-mechanical gate is estimated to be very high for the proposed pier top level and a single unit of vertical lifting gate is not possible over 20m height. So, it was decided to provide a breast wall on the flood regulator crest to contain the downstream water level to the required height.

The breast wall of the flood regulator generally consists of two parts viz., vertical stem and horizontal beam. The vertical stem is designed as a slab spanning between the piers or pier and abutment, fixed at the two ends and loaded by the horizontal water thrust on the downstream side. The horizontal beam is designed for bending and torsion. When the stem and beam are monolithic, the beam of the breast wall has to be checked for torsion caused by the water thrust. The configuration of breast wall is shown in **Figure 15.21.**

The design has been done complying with the Indian Standard Codal Provisions. The stem is designed to resist the moments in both horizontal and vertical directions. The beam has been checked for torsional moments also and suitably reinforced. The breast wall elements are modelled and analysed in Finite Element Analysis software. From the STAAD Pro results it is found that the vertical stem of 0.55m thick and horizontal beam of size 0.55 m x 0.8m (B x D) is to be provided to take care of the loads acting on them. Analysis has been carried out to meet the Strength and Serviceability criteria of Limit State Design.

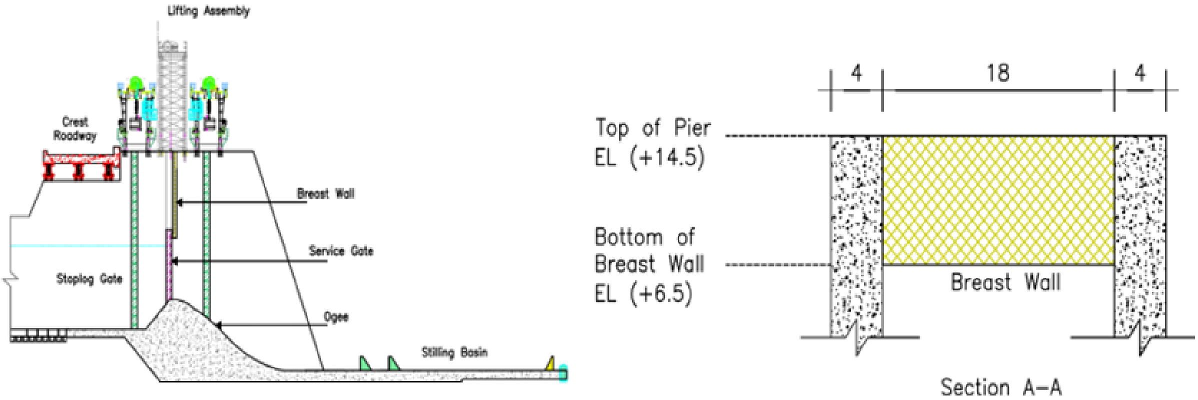

Figure 15.21: Configuration of Breast Wall

15.6.2 Ogee Weir

The ogee spillway has a control weir that is ogee-shaped (S-shaped)in profile as shown in **Figure 15.22**. Because of its high discharge efficiency, the nappe-shaped profile is used for most spillway control crests. On account of the geometry of the flood regulator crest profile, tensile stresses are developed in the crest because of the loads acting such as uplift pressures, water loads and silt pressures in both seismic and non-seismic conditions.

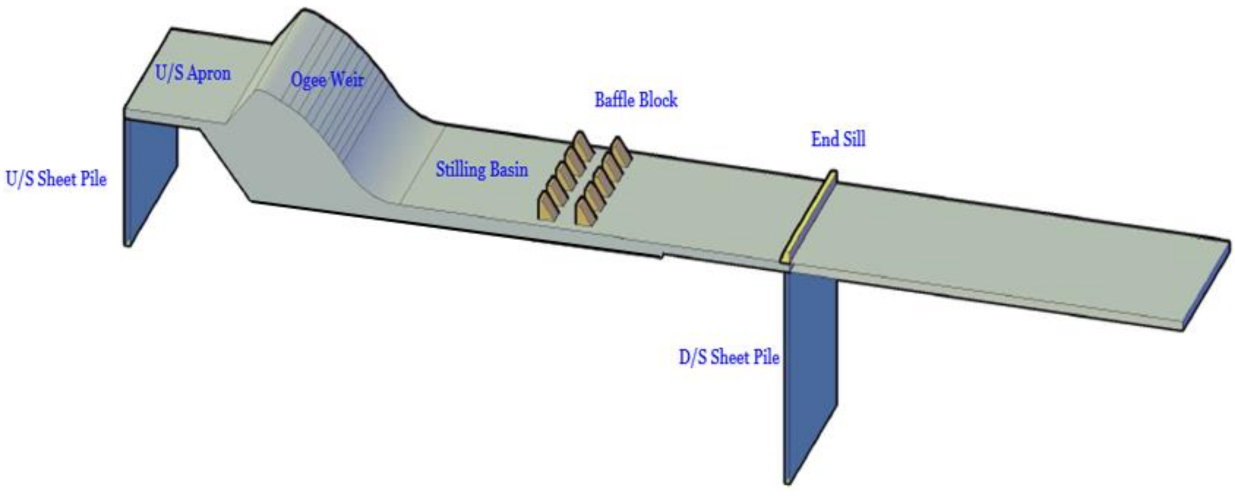

Figure 15.22: Configuration of Ogee Wier

The ogee weir is designed to meet the loads that exist at any time during the life of the structure. Design loads and load combinations are considered complying with the Indian Codal References Reinforcement needs to be provided to take care of these tensile stresses. The minimum thickness of structural concrete provided for the flood regulator crest is 1.5 m, measured normally.

From the tensile stress calculations, the maximum tensile stress obtained is within the permissible limits. To ensure better hydraulics performance in scouring, top 0.3m is proposed to have a high-performance concrete of M 50 grade.

15.6.3 Wall Piers

Wall Pier of 4m thick is proposed at about 99 nos. Wall pier in flood regulator experiences loads due to ogee weir, breast wall, vertical lift gates and their associated hydro-mechanical components and resists the uplift pressure. Wall Pier under the transportation system is supported on piles as shown in **Figure 15.23,** whereas near the ogee section it is supported on an RCC raft.

The pier on the flood regulator and the pier under the transportation system is connected through construction joints. The Wall pier on the flood regulator is designed as a plate element in staad pro and is analyzed for the loads and load combinations as per standard references. Soil conditions are modeled as elastic mat with sub-grade modulus. Wall Piers are analyzed for dry and flooded cases that occur during the operation of gates and corresponding wave and current loads are applied.

15.6.4 Retention Structures

(a)Abutment

An abutment is a substructure at the extreme ends of the flood regulator, which acts as an interface between the flood regulator and the dyke section. Abutment provides vertical and lateral support for the span of the flood regulator and acts as retaining walls to resist lateral movement of the rock fill of the bridge approach. The minimum width of the abutment provided is 4 m to meet the design requirements. The abutment is constructed monolithically with the raft and thus the moments due to earth pressure behind the abutment are transferred to the raft. The abutment design is done to meet the stability requirements.

(b)Flank Wall

On the upstream and downstream of the flood regulator, flank walls have been provided on both sides so that the entry and exit of the flow may be hydraulically efficient. A sketch of the Flank wall is shown in **Figure 15.24**. Abutments are vertical walls while guide bunds have a slope. Therefore, to transition the slope, flank walls are placed between guide bunds and abutments. The flank walls are provided with joints at suitable intervals and PVC water stoppers are in these joints.

(d) Wrap around Walls

Wrap around wall is designed as an interface between the abutment end with the channel and dyke rock embankment. The abutment is continued as a vertical retaining wall structure up to the end of stilling basin and Wing walls are projected at 45 degrees from the straight abutment to ensure a smooth hydraulic response at the end and it retains the rock fill embankment projected as a head structure at the end. The configuration of the Wrap around wall is shown in **Figure 15.25.**

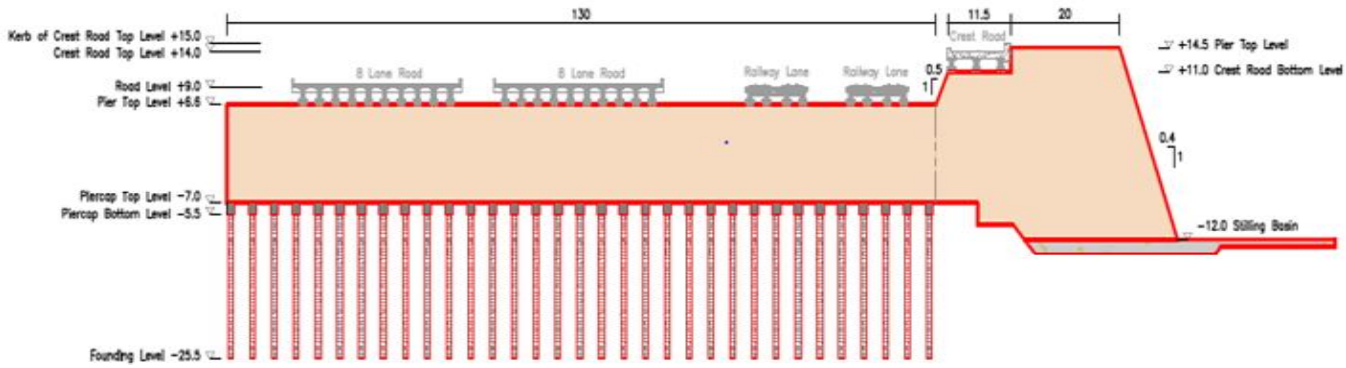

Figure 15.23: Configuration of Wall Pier

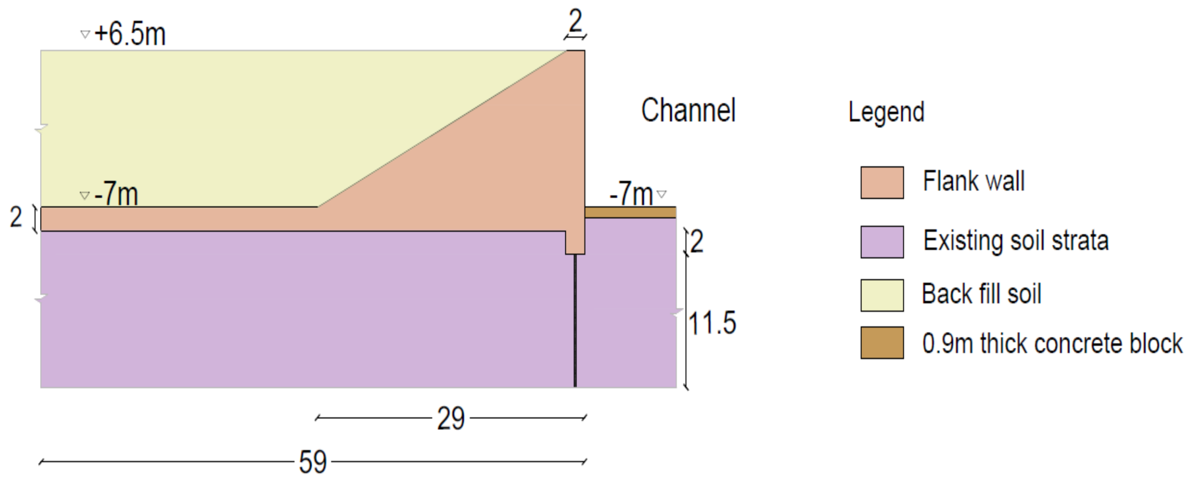

Figure 15.24: Configuration of Flank Wall

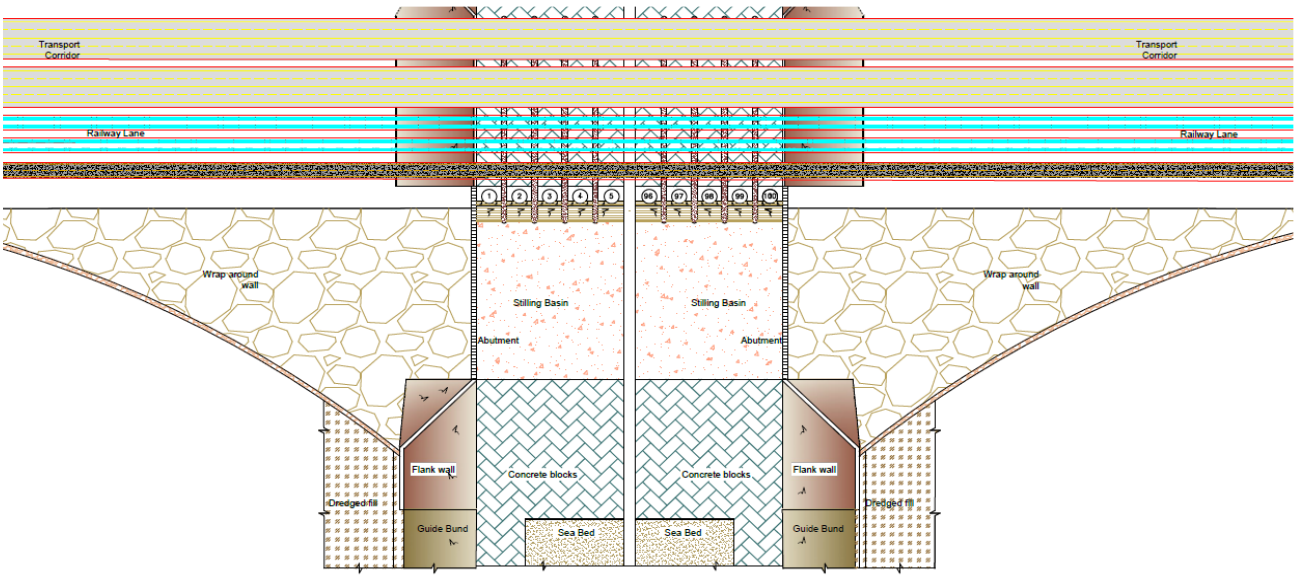

Figure 15.25: Configuration of Wrap around wall

15.6.5 Energy Dissipation Elements

Energy dissipating devices such as baffle blocks and end sill are provided on the stilling basin (downstream raft) as per the Hydraulic design proposal. The energy dissipation arrangement is shown in **Figure 15.26**.

The length of the raft on upstream is 26 m and downstream is 50 m, in addition secondary apron of length 50 m is provided considering the scouring action of high velocity flow. All these devices are designed complying with the Indian Standard Codal provisions. The raft of the flood regulator is designed treating it as a beam on elastic foundation as per theory. On a yielding foundation, it may suffer differential settlement, therefore the basin floor slab is designed to take care of the stresses induced on them.The location and optimum shape of baffle blocks and end sill are provided to meet the hydraulic design requirements. The baffle block and end sill are designed for the dynamic force acting on it due to the hydraulic forces on the downstream side.

15.6.6 Protection Works

The apron (RCC raft) length is computed in accordance with calculations for the exit gradient for seepage flow. Beyond the impervious foundation,previous protection comprising of cement concrete blocks and a launching apron of loose boulders or stones is provided to spread uniformly over scoured slopes. Where the stone is likely to be swept away due to high velocities or where somewhat smaller stones are to be used due to the non-availability of stonesof specified size, the loose stone apron should be provided in the form of wire sausages of suitable size.

The water seeping below the body of the hydraulic structure endangers the stability of the structure and may cause its failure, either by piping or by direct uplift. In order to overcome these failures and also to overcome the scouring action Sheet Piles are proposed on both the upstream and downstream sides of the flood regulator. Due to ease of construction and availability of work windows in gulf steel sheet piles are preferred. As per the Hydraulic design requirements, the length of sheet piles of 11.5 m and 12.5 m are provided upstream and downstream respectively.

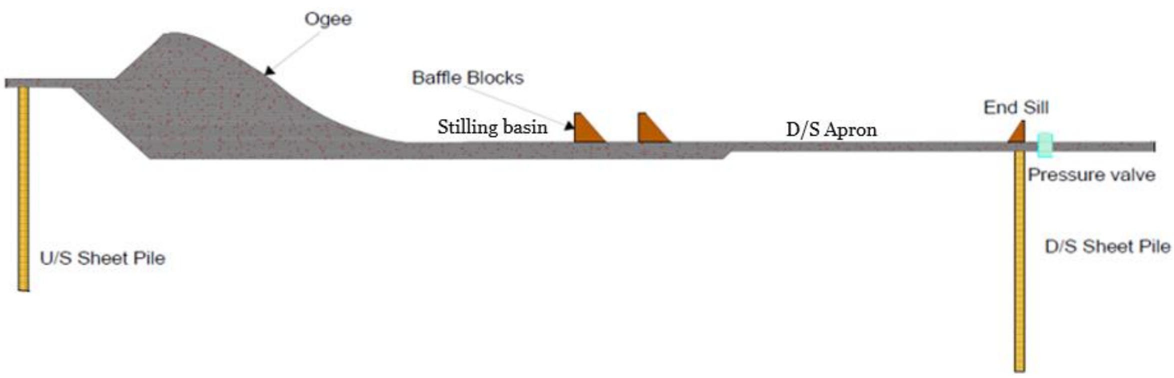

Figure 15.26: Energy Dissipation Arrangements

15.6.7 Recommendations

Following are the recommendations given from Structural aspects of the Flood Regulator.

(i) Ogee Control Structure has been recommended to regulate the flood from the reservoir. Top 1.75 m thick of the ogee weir is a Structural concrete. Reinforcements are proposed to meet the design requirements. Also, to ensure better hydraulics performance in scouring, top 0.3m is proposed to have a high-performance concrete of M 50 grade.

(ii) Breast wall of thickness 0.55m has been proposed between the piers to contain the downstream water level to the required height.

(iii) Wall Pier of 4m thick is proposed to support ogee weir, breast wall, vertical lift gates and its associated hydro-mechanical components.

(iv)Crest road is proposed on wall pier to inspect and operate gantry girders and hydro-mechanical gates. Suitable girders, cross beams and deck slab are proposed.

(v) Energy dissipating devices such as baffle blocks and end sill are recommended to dissipate the kinetic energy due to high velocity turbulent flow entering the basin.

15.7 Summary

A summary of hydraulic, geotechnical and structural aspects is listed as under:

15.7.1 Hydraulic Design

The hydraulic design of the approach channel has been done to maintain a uniform flow on the upstream channel. The control structure is designed to make the flow smooth without forming sub-atmospheric pressure which may result in cavitation problems. Energy dissipating elements like baffles and end sill are designed to dissipate high energy turbulent flow and make flow sub-critical. Uplift pressure calculation and sheet pile length calculation have been carried out using Koshla's theory.

15.7.2 Geotechnical Design

(i) The critical borehole locations with respect to the design are identified based on the available geotechnical data. The safety and stability of the floor regulator and protection works have been analysed. The bearing capacity in the depth of excavation is found sufficient to resist the loads from the structure. The settlement of the stilling basin along with the ogee weir is found within permissible limits;

(ii) The retention structure configuration is determined. Stability analysis is carried out for all retention structures against sliding and overturning for both static and pseudo-static cases. The eccentricity and bearing capacity of the proposed structures were also checked. The type of temporary measure to be adopted for the construction of flood regulator is also briefly explained.

15.7.3 Structural Design

The structural design of flood regulator components is designed to meet the structural loads acting on each component. Major loads would include dead load, live load, water load (static and dynamic), wave load, current load, wind load and earthquake load. Finite Element Analysis has been carried out for each component considering various loads and load combinations.

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