2.3
Flood Regulator Design

2.3.1 Past studies

National Centre 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 flood regulator, and the report prepared by Dr. R. M. Khatsuria (earlier with Central Water and Power Research Station, CWPRS) for preliminary design of the flood regulator. The location of the flood regulator has also been finalized earlier by the NCCR.

(1) Location of the Flood Regulator

It has been decided earlier by the NCCR to locate the flood regulator in intertidal region of Dahej as shown in **Figure A2.301**.

Figure A2.301: Location of flood regulator on Google map

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 approach channel and tail channel. As this alternative is located near to 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 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. However, this will require larger lengths of approach channel and tail channel, which would increase the dredging cost.

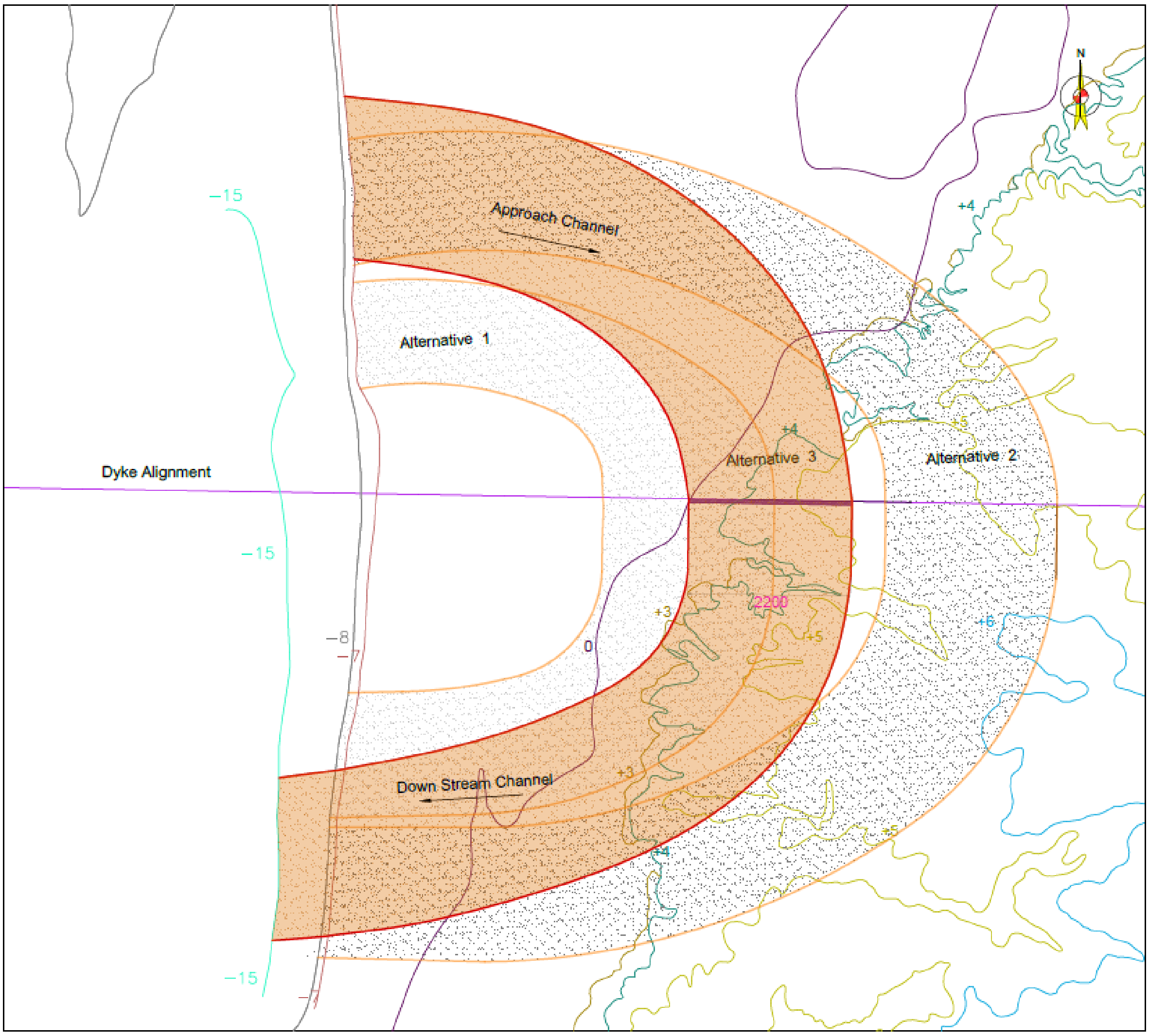


Figure A2.302: General Layout - Location proposal of Flood regulator (Plan)

Considering the merits and demerits of the two locations, a third alternative is proposed in between these two alternatives as shown in **FigureA2.302**. This alternative will start at 4m MSL which will have lesser wave impact than alternative I. Also, the smaller approach channel length, reduces the dredging quantity which reduces the dredging cost. Thus, alternative III has better advantage over alternative II and I.

(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 has 3 major river systems in north and east namely Sabarmati, Mahi, Dhadhar along with Narmada diversion canal and the 7 small rivers in Saurashtra to the west. However, it is assumed that water from 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. Probable Maximum Flood (PMF) for Kalpasar Reservoir was estimated through following steps:

(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 detailed summary of the above is covered in the volume 2 of Part H (Annexures)

(a) Estimation of Maximum Rainfall in Catchment Area

Standard Project Storm (SPS) is defined as the severe most rainstorm on record yielding highest rain depth over the catchment or in the meteorologically homogenous neighbourhood of the catchment. The available rainfall data from 1901 onwards in respective stations in and around the basin were examined. The following severe most rainstorms were considered in their study: (1) 26-28July 1927 (centre at Dakor) (2) 20-22June 1983 (Centre at Upleta) (3) 23-25July 1905 (Centre at Radhanpur).

Probable Maximum Precipitation (PMP) would be a result of combination of highest rain producing water in the environment. It is estimated that the highest rain producing efficiency is experienced during occurrence of most severe rainstorm over the basin. Hence SPS value is maximised by Moisture Adjustment Factor (MAF) for evaluation of Probable Maximum Precipitation.

The return period analysis for 500, 1000, and 10000 years of 1D, 2-D and 3-D annual maximum rainfall data sets for 128 rain gauge stations using Gumbel EV1 distribution using Probability weighted moments method was carried out. The results of frequency analysis of 1D, 2D, and 3D AMS indicate that the maximum of 1000 years return period rainfall are comparable with the observed maximum rainfall on any station in the catchment and in the storm used for the purpose of PMP computations. So, PMP is as Maximum rainfall in the catchment Area.

(b) Flood generated by each River Basin

Flood generated by a river basin at the Kalpasar lake is obtained by combining,

(i) Flood generated by dam release (routed one), ii) Flood generated by gauged basin (routed one) and iii) Flood generated by un-gauged basin. For Gauged stations, flood events which are not affected by dam releases on daily basis, and selection of up to 5 major flood events for carrying out detailed analysis. The Unit hydrograph are estimated based on Nash Model using Gauge Discharge Data. For ungauged Basins, Average dimensionless Unit hydrograph using the standard CWC methods. The unit hydrographs are combined along with PMP is used for obtaining the flood. The hydrological routing of the flood is done using Muskingum method.

(c) PMF at Reservoir by combining Flood from various Basins

The floods at the Kalpasar Lake have been computed for two scenarios viz. Storm centred over Dhadhar, Mahi and Sabarmati region and Storm centred over Saurashtra region. It is observed that storm centered over Dhadhar, Mahi and Sabarmati have maximum flow.

The PMF flood volumes in case of storm centering over Dhadhar, Mahi and Sabarmati have been taken for computations of design length of flood regulator. The length of flood regulator is estimated by solving the continuity equation. The inflow in equation is due to PMF and outflow from 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 meter and the tide is rising. The raise in reservoir level is limited to El +5.0m. Flood regulator having 95 spans of 18 meters width and 94 no. of 4 m thick piers with 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 totally 100 spans of 18m wide accordingly the net width and gross width corresponds to 1800m and 2196m respectively.

(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.

Alternative crest elevations of -3.5m, -2m, 0.0m 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 El +3.0m, the crest level being at El -3.5m. The maximum operating head corresponding to HFL of El +5.0m 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.

2.3.2 Design Criteria

Flood regulator is a 2.2 Km long gravity based rigid structure constructed on flat ground using RCC. Accordingly, the design basis for three components namely, Hydraulic, Geotechnical and structural design are finalized as below.

(1) Hydraulic Design

Hydraulic design of Flood Regulator components such as Approach Channel, Upstream Apron, Control Structure, Downstream Apron (Stilling Basin), Energy Dissipating elements, Sheet Piles and Protection works are to be designed to meet the fluid-structure interaction behaviour.

The Approach channel needs to be designed in such a way that it is desirable to meet the approach velocity on the upstream side of the flood regulator. Protection works will be designed to prevent scouring action on the upstream and downstream of the flood regulator.

Sheet piles and Apron (on upstream and downstream) are subjected to piping action and uplift pressure forces acting under the raft. Therefore, the hydraulic design of the sheet pile and apron need to be designed considering the aforesaid criteria.

The shape of the control structure has designed to comply with the smooth hydraulic flow profile over the control structure. When the high-velocity turbulent flow reaches the downstream, the energy has to be dissipated to make it a sub-critical flow. For this reason, energy-dissipating elements need to be designed.

(2) Geotechnical Design

Geotechnical analysis has been carried out for the safety and stability of the flood regulator need to be designed by considering dead load, imposed load, hydrostatic load, seismic loads, and surcharge and construction sequence as per relevant Indian Standards. The following criteria have been considered in the geotechnical analysis:

(1) Bearing Capacity

The safe bearing capacity at the founding level need to be estimated as per IS 6403:1981 with a factor of safety of 2.5. the bearing stress on the soil need to be analysed for various scenarios and ensure well below SBC. In addition, eccentricity check also needs to carry out to avoid loss of contact of foundation with founding soil layers.

(2) Settlement

Settlement is essential design criterion of the foundation to check the serviceability criteria. A raft foundation needs to be designed as foundation for flood regulator. The permissible settlements for raft foundation in the structure need to be considered as per IS 1904:2021. The same are shown in **Table A2.301**.

Table A2.301: Permissible Settlements

Type	Units	Sand and Hard Clay	Plastic Clay
Maximum Settlement	mm	75	100
Differential Settlement	mm	0.002 L	0.002 L
Angular Distortion		1/500	1/500

(3) Stability

The stability of the flood regulator for various loading scenarios needs to be computed as per relevant National Standards. The suitable factor of safety to be ensured to arrive at final configuration.

Table A2.302: Factor of Safety for Stability (for flank wall)

S. No	Type of Analysis	Required FOS
1	Static Sliding	1.75
2	Static Overturning	2
3	Dynamic Sliding	1.5
4	Dynamic Overturning	1.5

(4) Channel Protection Works

Suitable channel protection work needs to be provided both on the upstream and downstream guide bunds. The protection works prevent the erosion of the bund due to the flow velocities that occur in the channel. Suitable channel protection works like mattress need to be designed as per IRC: SP: 116-2018.

(3) Structural Design

The structural design needs to carried out for two critical scenarios namely, dry condition and flooded condition.

(a) Pier

(i) Dry Condition

Flood regulator gates are considered to be closed in this case and there is no water flow at this stage of analysis. Firstly, the pier structure is analyzed for static load with dead load, deck load, wind load, and loads due to breast wall and vertical lift cases. Analysis with the same loads is again carried out for dynamic cases in combination with seismic loads.

(ii) Flooded Condition

Hydro-mechanical gates are considered to be open and water flow will be there in bays. In addition to above mentioned static loads, water currents, hydrostatic loads and buoyant uplift forces are considered for this analysis. The same load condition is checked for a combination of seismic loads along with hydrodynamic loadings. Above mentioned dry condition and the flooded condition are again carried out for

(1) Minimum drawdown on the Upstream side and High tide level on the downstream side; and
(2) Full Reservoir level on the upstream side and Low tide level on the downstream side

(b) Crest Road

Crest road is designed to carry the Imposed live load caused by vehicles during the operation and maintenance of Hydro mechanical gates, as per IRC-6 2014.

(c) Scour Protection

(i) Stilling Basin

The uplift pressures need to be calculated as per Khosla's theory, for the permeable foundation. The thickness of the floor has been determined for both maximum static conditions and dynamic conditions. The higher value (thickness) is adopted in the design.

(ii) Cut-off Wall

The upstream and downstream cut-offs should generally be provided as 1.5 D and 2.0 D respectively, where D is the normal scour depth as per Indian Standards. However, the factor of safety is not provided because the channels on both the upstream and downstream sides are lined with concrete. The cut-off should be suitably extended into the banks on both sides up to at least twice their depth from the top of the floors.

(iii) Energy Dissipation Elements

The location and optimum shape of baffle blocks need to be decided on the basis of IS: 4997-1968. The baffle block is designed for the dynamic force acting on it due to the hydraulic forces on the downstream side.

(iv) Protection Works

The length of upstream block protection is provided equally to D (the design depth of scour below the floor level) and 1.5 D on the downstream. The length of loose stone protection shall be 1.5 D to 2.5 D.

(d) Breast Wall

The beam of the breast wall spanning between the pier and abutment/pier has been designed to resist the moments due to: a) Dead load of the breast wall; b) Uplift; c) Water pressure; d) Seismic forces and moments, if any; and e) Hydrodynamic forces due to seismic conditions. The beam and stem are designed to resist the moments in both horizontal and vertical directions. The beam needs to be checked for torsional moments and suitably reinforced.

(e) Energy Dissipation

Hydraulic jump results when there is a conflict between upstream and downstream controls which influence the same reach of the channel. For example, if the upstream control causes a supercritical flow while the downstream control dictates a subcritical flow, then there is a conflict that can be resolved only if there are some means for the flow to pass from one flow regime to the other. The phenomenon of jumping of water from supercritical flow to sub-critical flow is known as Hydraulic Jump (HJ). Hydraulic jump is generally accompanied by large scale turbulence, dissipating most of the kinetic energy of supercritical flow which has got a detrimental effect on the surface. For a structure to be safe, the formation of a jump should be confined to the sloping glacis and not allowed to be formed on the cistern level beyond the toe of the glacis. The cistern level and its length are to be worked out for various sets of conditions imposed on the structure on the basis of the gate regulation proposed. The most critical condition gives the lowest cistern level and its length. These are generally determined by the use of curves available for various discharge intensities and water depths or by analytical methods.

There should be an energy dissipation arrangement in order to make sure that the hydraulic jump forms stably within the stilling basin. The energy dissipation arrangement consists of providing one or more rows of baffle blocks and an end sill. Sizes and spacing of these elements are based on standard designs evolved in the past by various well-known research organizations such as the United States Bureau of Reclamation (USBR), U.S. Army Corps of Engineers, etc.

2.3.3 Configuration

Unlike normal flood regulator, the flood regulator in our case satisfies two purposes. Not only will it let off the excess flood water into the sea, but it will also prevent flow of sea water when water level rises during storm surge, thus preventing increase in salinity of the reservoir. The tidal variations on the seaside are estimated to be +6.2m and -5.3m MSL post-implementation.

Location of the flood regulator shown in **Figure A2.303** has been proposed considering the cost of dredging and the reduced effect of wave action on the structure. The arrangement proposed for flood regulator section has been designed as a standard WES elliptical profile for a design head of 6.5 m corresponding to FRL El +3.0 m, the crest level being at El -3.5 m. The maximum operating head corresponding to HFL of El +5.0 m would thus be 8.5 m. The span width would be 18 m, separated by 4 m 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 -7.0 m 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 -12.0 m.

The channel of width 2200m is extended on both upstream and downstream of flood regulator to meet El (-) 7 m contour and El (-) 15 m contour respectively.

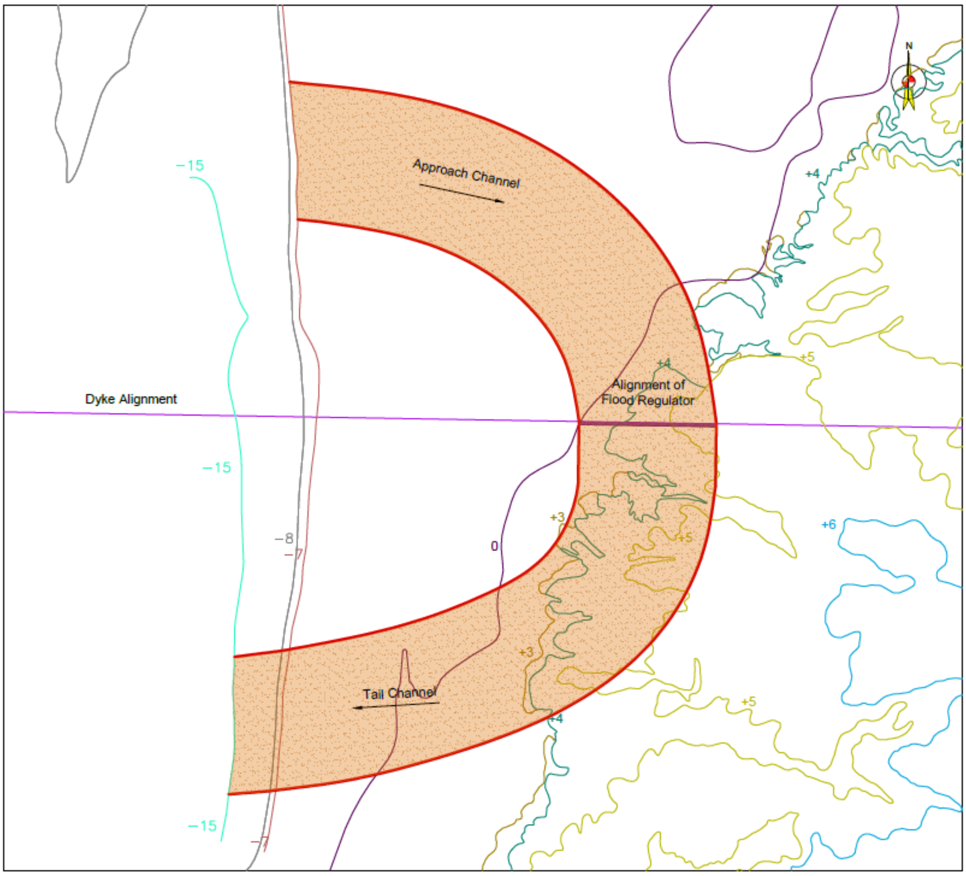


Figure A2.303: Schematic View on Location of Flood Regulator

The overall geometry of the flood regulator is shown in **Figure A2.304**. The major components of the flood regulator are:
(1) Ogee Weir;
(2) Stilling Basin;
(3) Energy Dissipation Arrangement;
(4) Spill Channel; and
(5) Protection Works.

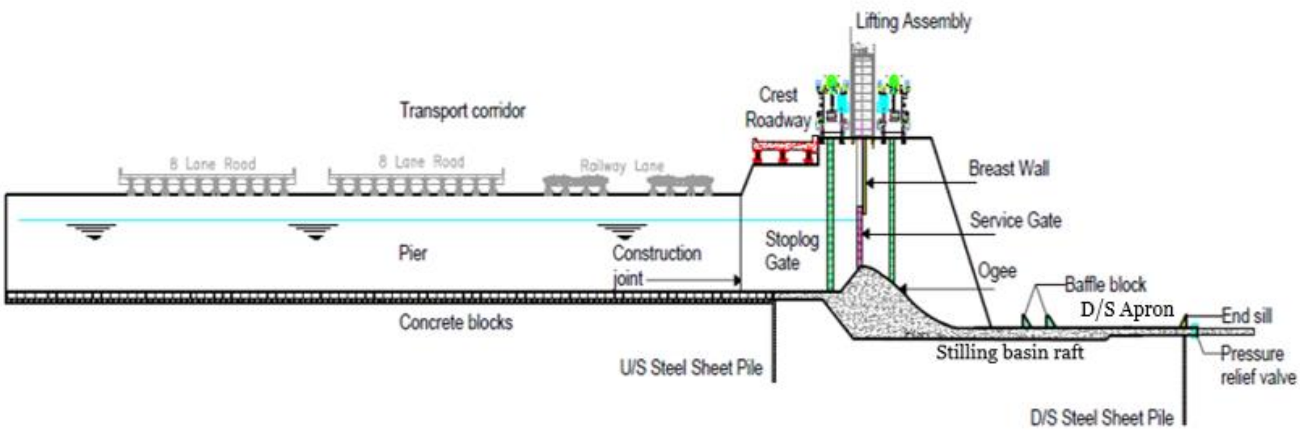


Figure A2.304: Schematic cross section of the Flood Regulator

(1) Ogee Weir

Based on the site conditions and the economics, an ogee type of flood regulator is chosen. A typical cross-section of the flood regulator is shown in **Figure A2.305**.

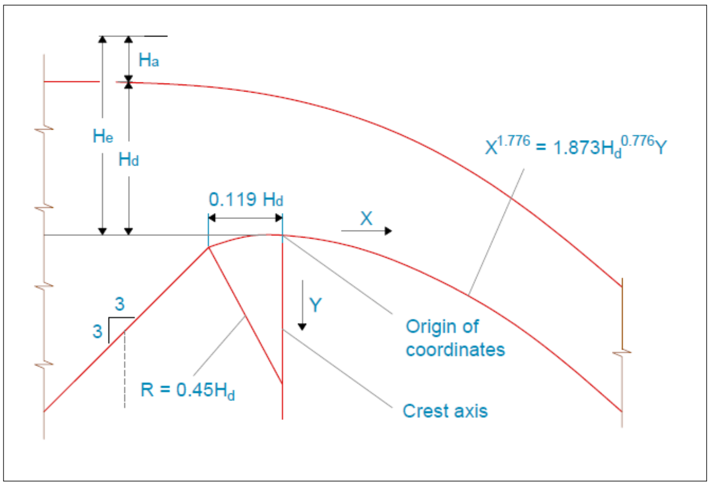


Figure A2.305: Typical shape of the Ogee Weir

The shape of the crest or the upper curve of the ogee profile is made to conform closely to the profile of the lower surface of the nappe (or lower nappe) or sheet of water flowing over a ventilated sharp-crested weir. This shape of the crest profile provides high discharge efficiency without causing cavitation. The general equation for the shape of the profile downstream of the crest i.e., the origin is:

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

where, x and y are distances in X and Y directions; H_d is the design energy head above the weir crest and, K and n are empirical coefficients. K and n depend upon the upstream slope of the flood regulator. The shape of the curve between the upstream slope and the crest (on the upstream side of the crest) is chosen to increase the coefficient discharge of the weir.

Thus, ogee weir is proposed as a control structure to control the flow of reservoir water from upstream to downstream. The proposed span of ogee between pier is 18m. The crest of ogee is fixed at El (-) 3.5 m. The profile is designed to conform with the equation mentioned above.

(2) Stilling Basin

The function of the stilling basin on the downstream side of the weir is to create conditions for the formation of a hydraulic jump which dissipates the high energy of the incoming flow. Erosion of the downstream channel would occur if this energy is not dissipated. The floor level of the stilling basin should be chosen such that the hydraulic jump forms within the stilling basin or on the downstream sloping face of the weir. This will happen when the depth of water in the spill channel just downstream of stilling basin (i.e., at the upstream end of the spill channel) is more than or equal to the depth of flow downstream of the hydraulic jump. The worst condition for which design needs to be carried out is: flow is equal to the design flow of 110,000 m^3/s and LTL at the end of the spill channel i.e., at the confluence of the spill channel and the estuary. This is shown in **Figure A2.306**. Therefore, the design of stilling basin is tied to the design of the spill channel.

The length of the stilling basin should be longer than the length of the jump. The thickness of the stilling basin should be such that the weight of the stilling basin floor is more than the net uplift force. The net uplift force is equal to the uplift force coming on the stilling basin from below due to seepage flow minus the weight of water above the stilling basin if any. The uplift forces are considered for two cases: (i) water level on the upstream side of the weir is equal to the FRL (+3.0 m), there is no flow and the downstream water level corresponds to the low tide level (-5.3 m) and (ii) passage of design flood of 110,000 m^3/s, with upstream water level equal to (+5.0 m) and downstream water level corresponding to the water level downstream of the jump. Uplift pressures are determined using the widely accepted Khosla's theory for seepage underneath hydraulic structures.

Stilling basin of length 50 m is proposed on downstream of flood regulator to dissipate the high energy, in addition a stretch of 50 m secondary apron is provided on safety aspects. Thus, a total length of 100m is provided on downstream of the flood regulator. The concrete floor thickness shall be 2.5 m for the entire length of stilling basin (downstream). To counteract the uplift pressures, bored cast in-situ piles are to be provided.

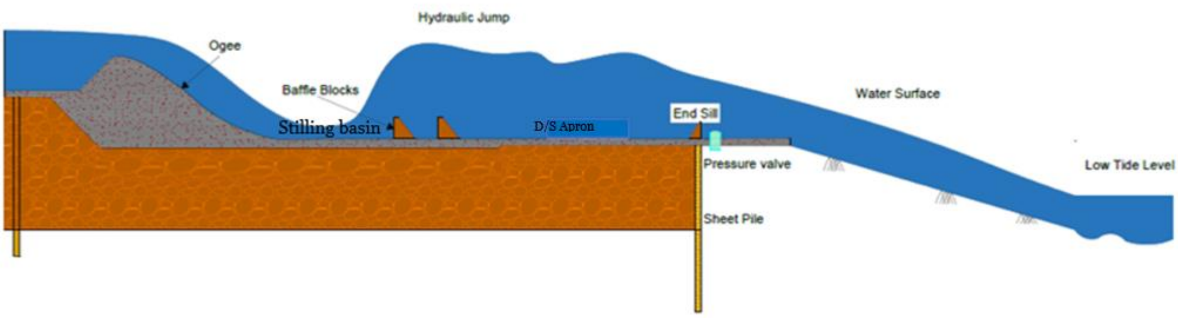


Figure A2.306: Occurrence of Hydraulic Jump in Stilling Basin

(3) Spill Channel

The function of the spill channel is to carry the flood from the flood regulator to the estuary on the downstream side. The bed slope and the roughness of the spill channel are chosen such that the depth of the flow in the spill channel at its upstream end should be greater than or equal to the depth of flow downstream of the hydraulic jump. It should be noted that the flow need not be uniform in the entire spill channel and for all flow conditions. Gradually varied flow conditions will exist in the spill channel. The design of the spill channel is based on Manning's equation for the slope of the energy grade line S_f.

$$S_f = \frac{Q^2 n^2}{A^2 R^{1.33}}$$

where, Q = discharge; n = Manning's roughness coefficient; A = flow cross-sectional area and R = hydraulic radius. The cross-sectional shape of the channel is trapezoidal, with side slopes being 1H: 1V. The gradually varied flow profile can be described using the following equation to determine the flow depth variation in the channel.

$$\frac{dh}{dx} = \frac{S_0 - S_f}{1 - \frac{Q^2T}{gA^3}}$$

where, S_o= bed slope; g = acceleration due to gravity and T = top width. In the case of wide channels (the width of the channel is much larger than the depth), the above equations simplify to

$$S_f = \frac{q^2n^2}{h^{4.33}}$$

$$\frac{dh}{dx} = \frac{S_0 - S_f}{1 - \frac{q^2}{gh^3}}$$

where, h = flow depth and q = discharge per unit width. Gradually varied flow equations can be solved using the direct step method (an easy method for backwater calculations), starting at the downstream end.

Thus, Spill channel of width 2200m is provided on downstream to form a downward gradient from stilling basin level El (-) 12.0 m to sea bed level El (-) 14.5 m along a stretch of 10 km.

(4) Energy Dissipation Arrangement

There should be an energy dissipation arrangement, as shown in **Figure A2.307** in order to make sure that the hydraulic jump forms stably within the stilling basin. The energy dissipation arrangement consists of providing one or more rows of baffle blocks and an end sill. Sizes and spacing of these elements are based on standard designs evolved in the past by various well known research organizations such as the United States Bureau of Reclamation (USBR), U.S. Army Corps of Engineers etc.

Baffle blocks are provided in two rows of which the first row starts at 19 m from toe of ogee weir and the second row starts at a distance of 5.3 m from the face of first row. The width and spacing between each block in transverse direction is 2 m. The end sill is proposed at the end of first stretch of 50 m of stilling basin.

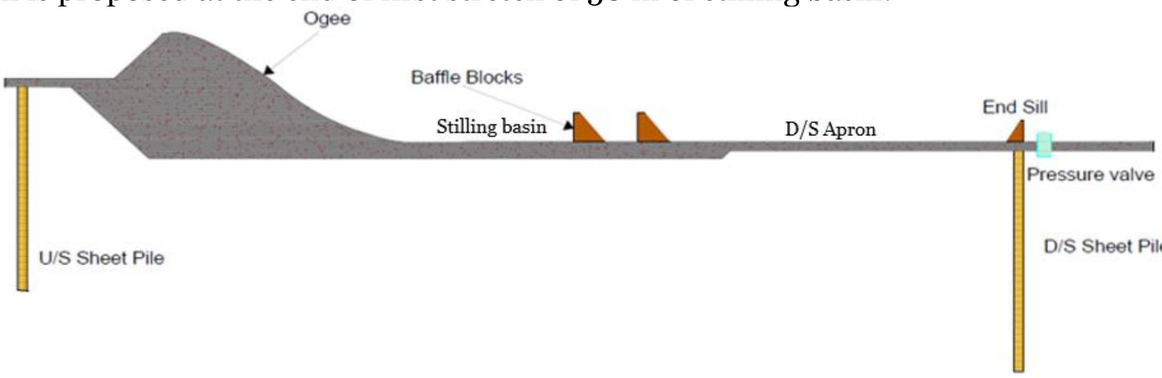


Figure A2.307: Schematic diagram of the Energy Dissipation Arrangement

(5) Protection Works

The flood regulator should be protected from failure due to any possible scour on the upstream and downstream sides. The protection works in the form of upstream sheet piles and downstream sheet piles are provided as shown in **Figure A2.307**. The depth to which sheet piles should be taken is based on the expected scour. The scour depth can be estimated using Lacey's theory which has been widely accepted in India and recommended by the Indian Standards. Typically, a factor of safety of 1.25 is applied to the upstream sheet pile and 1.5 is applied to the downstream sheet pile. In the case of alluvial channels, further upstream and downstream protection works beyond the stilling basin floor are suggested in the form of placing concrete blocks and loose stone aprons.

Sheet Piles of length 11.5 m and 12.5 m is proposed on upstream and downstream end of flood regulator respectively to protect against the scouring action.

2.3.4 Components

The plan and longitudinal sections of the entire flood regulator and its components are presented in **Figure A2.308, Figure A2.309&Figure A2.310.**

The components that are designed as a part of flood regulator:

(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.

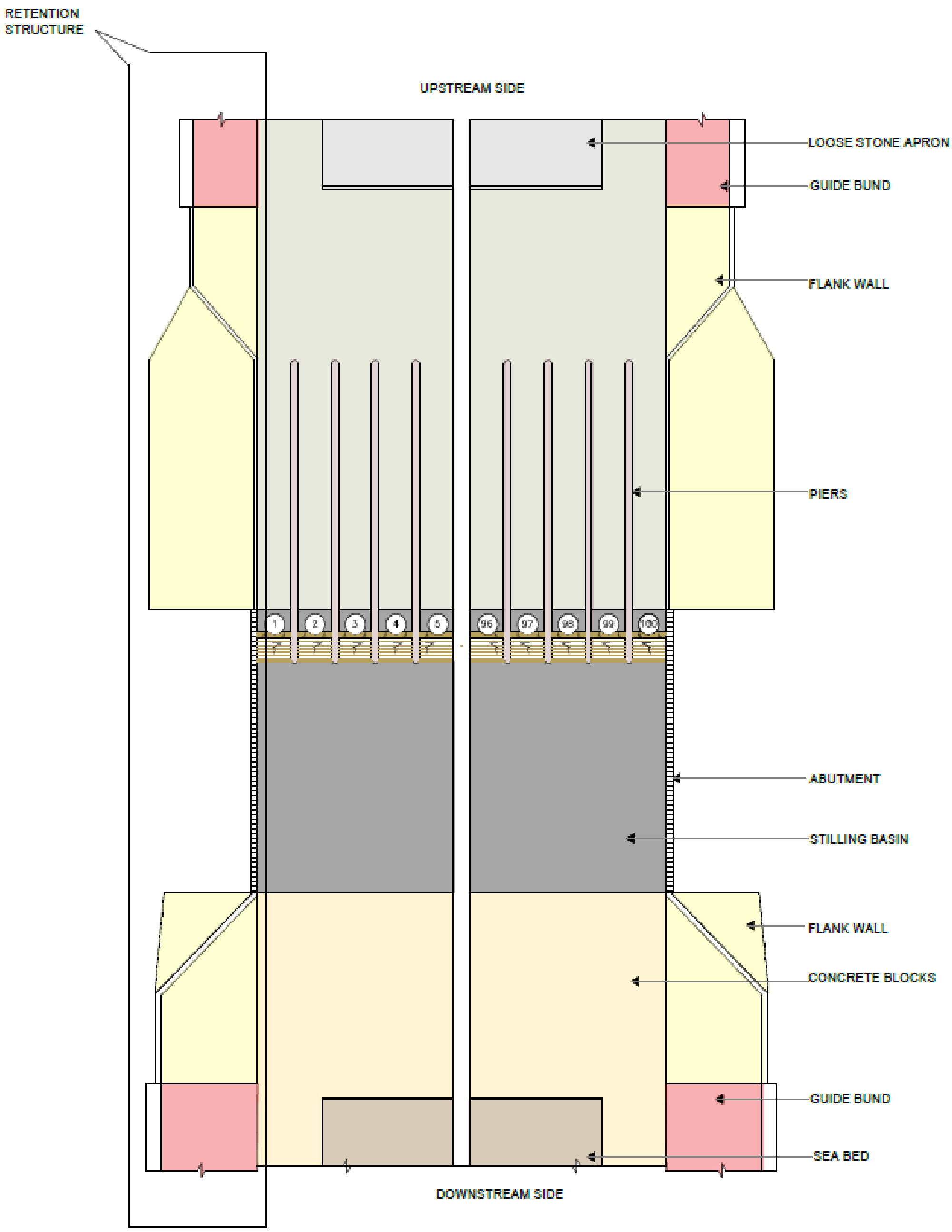


Figure A2.308: Plan of Flood Regulator

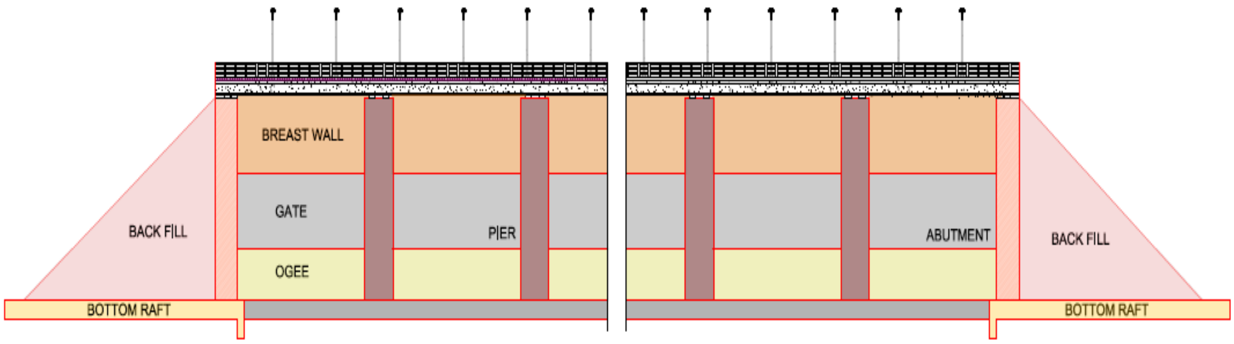


Figure A2.309: Longitudinal Section of Flood Regulator at ogee part

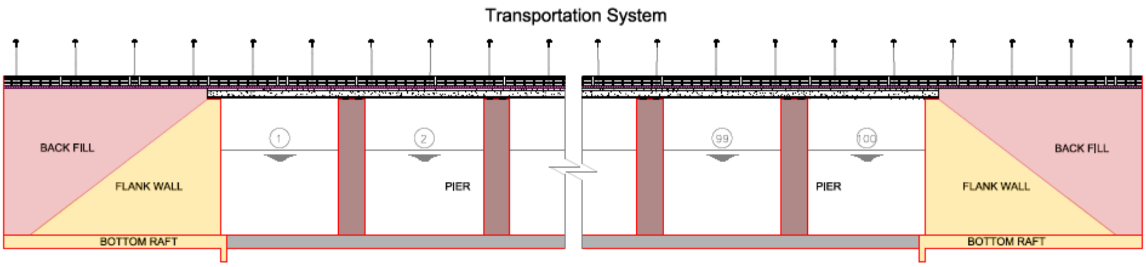


Figure A2.310: Longitudinal Section of Flood Regulator at flank wall

2.3.5 Waterway

A waterway of 2200 m is provided for the flood regulator. In this section, we present the details of calculations made to check the adequacy of the provided waterway. The schematic of the ogee weir is shown in the **Figure A2.311**.

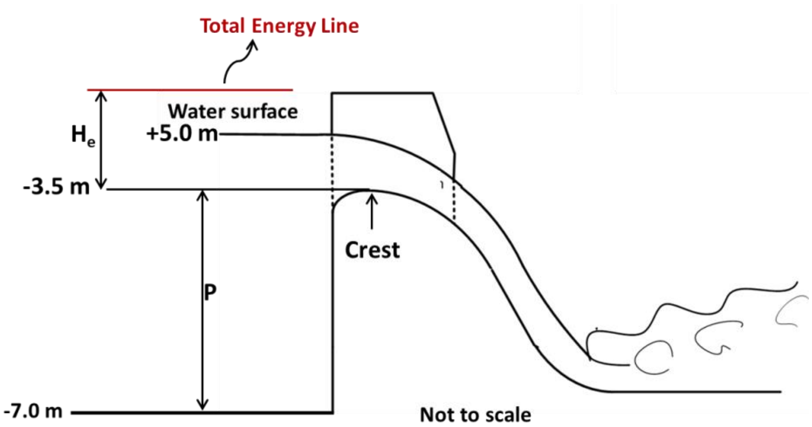
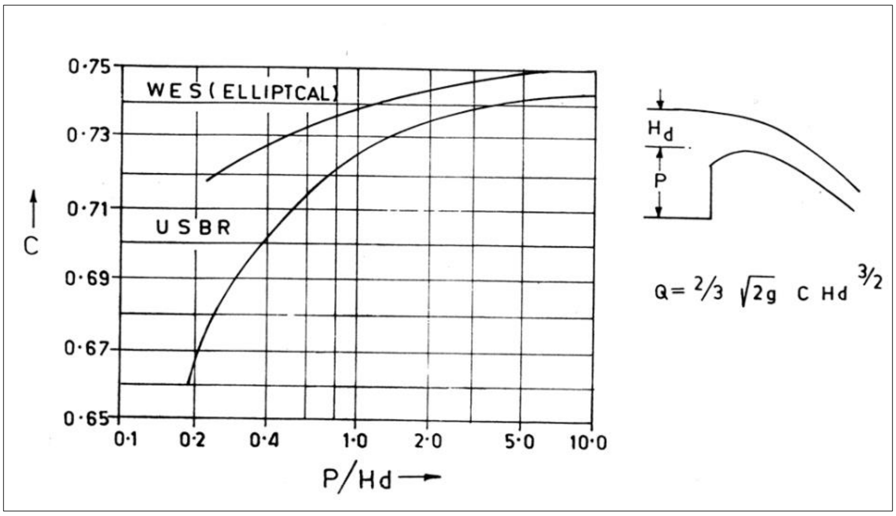


Figure A2.311: Schematic of the ogee weir

Design discharge, Q = 110,000 m^3/s
Crest level of the weir = -3.5 m
HFL = +5.0 m
Design head: h = 5+3.5 = 8.5 m
Bed level of upstream channel = -7.0 m
Height of weir = P = 7.0-3.5 = 3.5 m
Approach velocity $= \frac{110{,}000}{12 \times 2200} = 4.167\,\frac{m}{s}$
Velocity head = 0.884 m
Design energy head = H_e = 8.5 + 0.884 = 9.4 m
P/H_e = 0.37

We have designed the ogee weir as per the shape suggested by the Waterways Experiment Station (WES).

For this value of P/H_e; C = 0.726 (if upstream face = vertical) from **Figure A2.312**.



FigureA2.312: Effect of Approach depth on Coefficient of Discharge.

The suggested ogee weir has a 1:1 upstream face slope. Applying correction factor for a 1:1 sloping upstream face; (From Vente Chow's book; Fig. 14.4)

$P/H_d = 3.5/8.5 = 0.41$; Correction factor = 1.016; C = 0.738

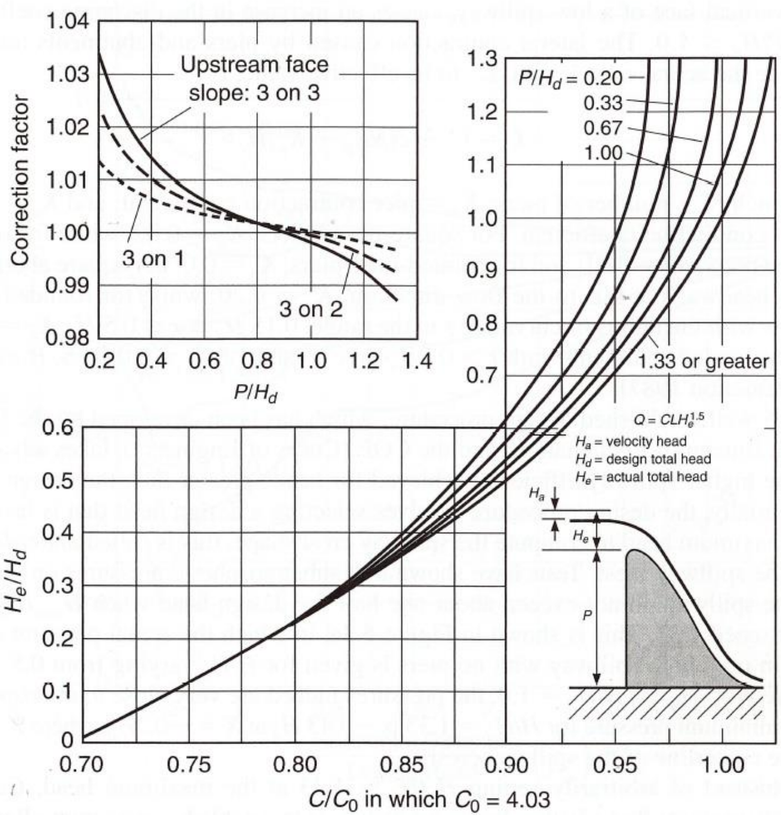


Figure A2.313: Correction Factor for Coefficient of discharge for an ogee weir with sloping upstream face

Total waterway width = 2200 m

Number of bays Provided = 100

Width of gate = 18 m

Clear water way, L = 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

Effective length, $L_e = L - 2 \times (N \times Kp + Ka) \times H_e = 1761\ m$

$Q = \frac{2}{3} C.\sqrt{2g}.L_e.{H_e}^{1.5}$ = 110,581 m3/s > 110,000 m3/s

Therefore, provided waterway is adequate.

Provide 100bays of 18m wide waterway with 99 piers of 4m wide.

2.3.6 Hydraulic design

(1) Approach channel

Approach flow depth in front of ogee weir to carry a design discharge Q is 110,000 m3/s is equal to 12.0 m. Therefore, the approach channel is designed to carry a Q of 110,000 m3/s at a normal depth of 12.0 m. The approach channel is designed as a concrete lined channel. It has a trapezoidal cross section with a bottom width is 2200 m and side slopes shall be provided as per geotechnical design,

The channel width B = 2200 m
Normal depth h = 12.0 m

Therefore, channel can be treated as a wide channel.

Manning's equation for a wide channel is:

$$Q = \frac{1}{n} B.h^{1.667}\sqrt{S_0}$$

Where
n = Manning roughness coefficient;
S_0 = bed slope;
Manning roughness coefficient n is taken as 0.014 for concrete; and
S_0 = 0.00012.

Provide a slope of 0.00012 to the approach channel.

The approach channel is concrete lined up to a length of 2.0 km on the upstream side of the flood regulator. Beyond this point, the channel is dredged into the reservoir. Water will enter from all the sides, when the reservoir level is +5.0 m.

Bed elevation of approach channel at a distance of 2.0 km

= -7.0 + 0.00012 x 2000 = -6.75 m

Water surface elevation in approach channel at a distance of 2.0 km

=5.0+0.00012 x 2000 = 5.24 m

Velocity in the approach channel = v = 110,000/ (2200 x 12) = 4.17 m

Consider the flow into the channel entrance as shown in **Figure A2.314**,

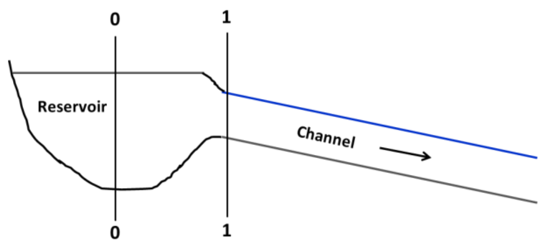


Figure A2.314: Schematic of flow into channel from the reservoir

Consider the sections 0 and 1 and apply the energy equation. Velocity head in the reservoir is negligible. Neglect losses.

$$WL_0 = WL_c + \frac{V^2}{2.g}$$

From the above equation, $WL_0 = 5.24 + \frac{4.17^2}{2\times 9.81} = 6.1\ m$

Thus, it is expected that the water surface elevation in the reservoir will be approximately at +6.10 m.

The discharge that can be passed over the ogee weir, without water level in the reservoir crossing +5.0 m = 87,000 m^3/s

To pass a flood of 110,000 m^3/s over the spillway, with a design head of 7.5 m (H_e = 8.0 m), and taking C = 0.738

$Q = \frac{2}{3} C.\sqrt{2g}.L_e.{H_e}^{1.5}$; Le = 2230 m.

This will require provision of total number 126 bays, each of 18 m span.

(2) Shape of the ogee weir

The IS code for spillways IS 6934 (1998): Recommendations for hydraulic design of high ogee overflow spillways [WRD 9: Dams and Spillways] is valid for high head spillways. 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. Therefore, we have adopted the design from well accepted practice worldwide. We are adopting the Waterways Experiment Station (WES) spillway shape with 1:1 upstream face. Details of WES spillways are provided from the book Open Channel Hydraulics by Ven te Chow.

A weir with upstream slope is chosen to get a higher coefficient of discharge, and consequently reduce the head required to pass the design flood over the weir. The weir shape has been arrived at based on WES recommendations to avoid cavitation and obtain a higher value of C. The weir shape is shown in the **Figure A2.315**.

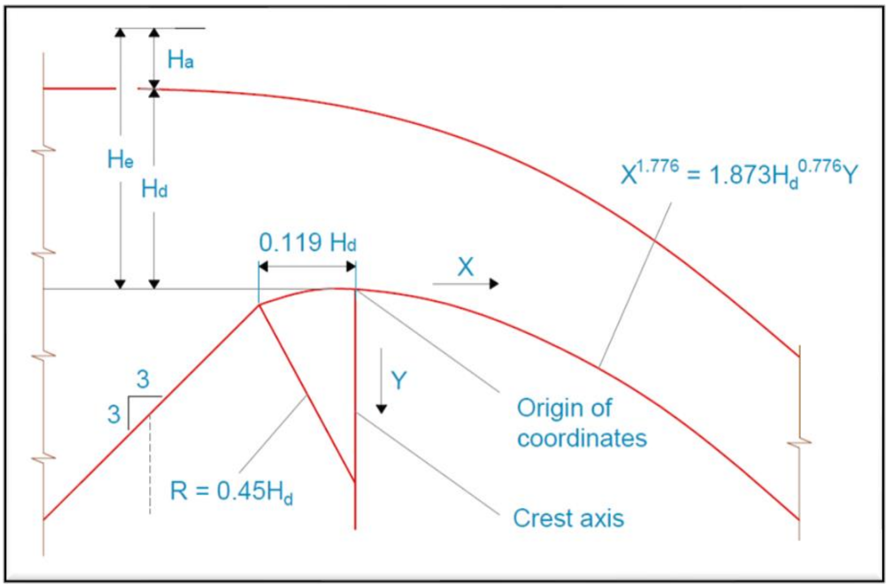


Figure A2.315: Shape of the Ogee Weir

The downstream face i.e., to the right of origin O; the equation for face is given as:

$$X^{1.776} = 9.857 \times Y$$

This curve joins smoothly to the cistern bed level at -12.0 m.

The radius of upstream curve = R = 3.825 m.

(3) Floor level of the energy dissipater

a) 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 same was suggested in an earlier report prepared by Dr. Khatsuria.

b) Fixing the Cistern Floor Level:

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 (**Figure A2.316**) for the energy dissipater.

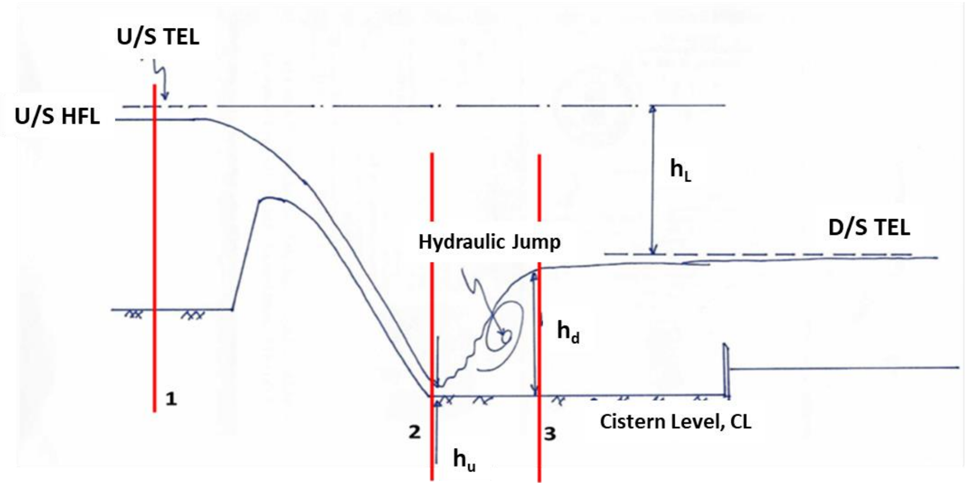


Figure A2.316: Schematic section for determining the Cistern Level of Energy Dissipater

In the above figure:

USHFL = High Flood level on upstream side = + 5.0 m
DSTWL = Tail water level on downstream side
h_u = Pre-jump depth
h_d = Post-jump depth
CL = Cistern floor level

Applying hydraulic jump relationship between Sections-2 and 3

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

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

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

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

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

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

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

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

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 the worst-case scenario.

Case-1: Design the cistern for a design discharge of 110,000 m³/s and tail water level corresponding to lowest tide level i.e., -5.3 m MSL

Design discharge = 110,000 m^3/s
Clear water way = 1760 m
Intensity of discharge, q = 110,000/1760 = 62.5 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 = 62.5/12 = 5.2 m/s
Velocity head = $\frac{5.2^2}{2\times 9.81} = 1.4\ m$
USTEL = 5+1.4 = 6.4 m
DSTWL = -5.3 m
Head Loss, h_L = 6.4+5.3 = 11.7 m

For q = 62.5 m^2/s and h_L=11.7 m; solution of equations (1), (2) & (3) will give:
h_u = 2.8 m; h_d = 15.52 m ; CL = -21.8 m

<u>Fixing of the cistern floor level at -22.0 m, while the average ground level varies from -1 to +4; could pose serious construction problems as well as it may lead to too much of cost.</u>

Several trials were made with other combinations of discharge and downstream water levels to fix the cistern level at a higher elevation. It may be noted that fixing up cistern level at any level higher than -21.8 m will result in the sweeping of hydraulic jump towards the downstream side if the PMF occurs during the low tide.

This issue was resolved by conjunctively designing the energy dissipater and the spill channel. An energy dissipater design suggested by the US Army Corps of Engineers is chosen. The schematic of energy dissipater is shown in **Figure A2.317**. The energy dissipater has two rows of baffle block and an end sill to restrain the movement of hydraulic jump due to variations in flow rate and the tail water level.

Schematic of occurrence of hydraulic jump at the tow of the ogee weir, along with the flow conditions in the downstream spill channel are shown in **Figure A2.318**.

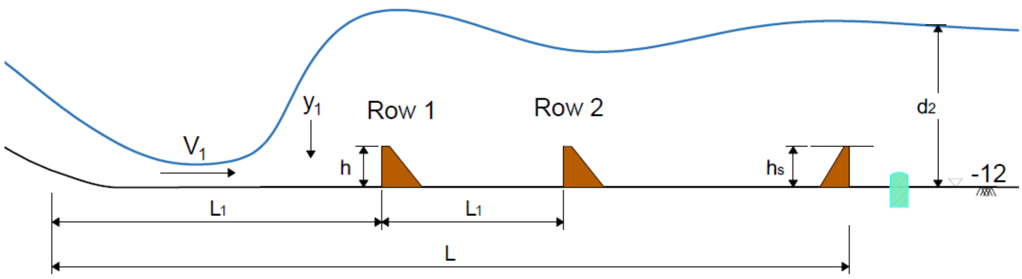


Figure A2.317: Schematic diagram showing the Energy dissipater

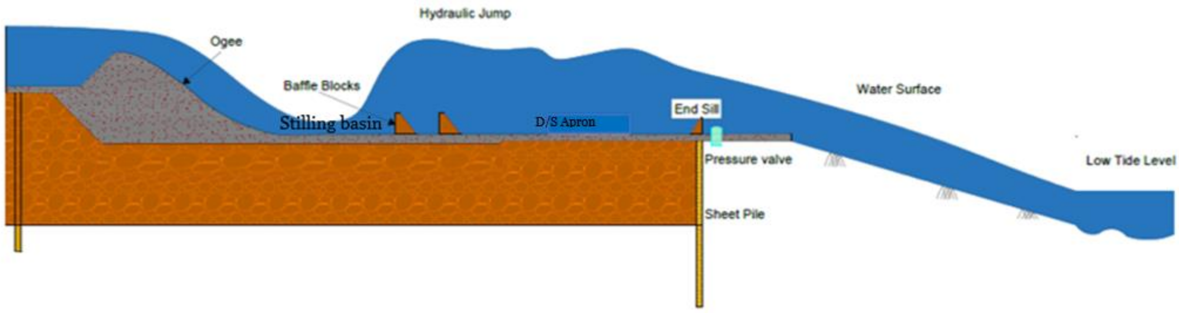


Figure A2.318: Schematic diagram of the flow in the Energy Dissipater and the Spill Channel

The energy dissipater and the spill channel are designed such that:

(i) The hydraulic jump forms at the toe of the ogee weir for a Q = 110,000 m^3/s;

(ii) The flow depth on the downstream side of the hydraulic jump matches with the flow depth in the spill channel at the entrance to the channel; and

(iii) The water level at the downstream end of the spill channel matches with the low tide level of -5.3 m.

After very many trials, following design parameters are arrived at:

The cistern level of the energy dissipater	= -12.0 m;
Length of the spill channel	= 10.0 km;
Slope of the spill channel	= 0.00025;
Width of the spill channel	= 2200 m;
Manning roughness coefficient of the spill channel	= 0.0175.

Calculations presented below show that the hydraulic jump forms at the toe of the ogee weir.

Q = Design flood = 110,000/1762 = 62.4 m^2/s

Assume energy loss on spillway face = 0.5 m

Approach velocity = 62.4/12 = 5.2 m/s

TEL on the upstream side = 5.88 m
TEL at the entrance to the stilling basin = TEL1= +5.88 – 0.5 = 5.38 m

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

Corresponding to design flood;

$$5.38 = -12.0 + y1 + \frac{62.4^2}{2.0.g.y1^2}$$

In which y1 = pre-jump depth.

y1 = 3.827 m
V1 = 16.3 m/s
Froude Number = F1 = $\frac{V1}{\sqrt{g.y1}} = 2.66$
Sequent depth y2 = $y1 \times \left(\frac{\sqrt{1+8F1^2}-1}{2}\right) = 12.61\ m$

For the USACE energy dissipater,

Depth downstream of the dissipater = 0.85 x y2 = 10.72 m

Flow conditions in the channel are now checked:

S_o = 0.00025; n = 0.0175; q = 110,000/2200 = 50 m^2/s

Critical depth = $h_c = \left(\frac{q^2}{g}\right)^{0.333} = 6.34\ m$

Normal depth = $h_n = \left(\frac{q.n}{\sqrt{S_0}}\right)^{0.6} = 11.1\ m$

As can be seen from **Figure A2.319** that an M2 type water surface profile will occur in the channel.

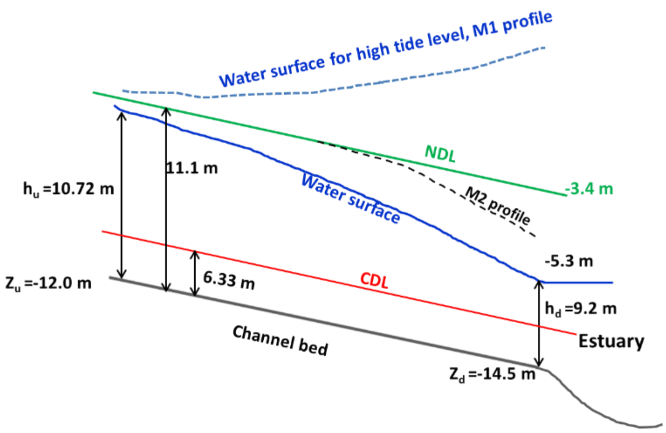


Figure A2.319: Water Surface Profile in the Spill Channel for Q = 110,000 m^3/s

Upstream bed level = -12.0 m. Downstream bed level (-14.5 m) and water level are fixed based on the slope of the channel and the low tide level of -5.3 m. Thus, the downstream depth = h_d = 9.2 m.

For a flow rate of q = 50 m^2/s and downstream depth, h_d= 9.2 m, we determine at what distance upstream a depth h_u = 10.72 would occur. For this purpose, we apply energy equation as follows:

$$Z_u + h_u + \frac{q^2}{2g\times {h_u}^2} = Z_d + h_d + \frac{q^2}{2g\times {h_d}^2} + \frac{\Delta x}{2}(Sf_u + Sf_d) \qquad (5)$$

Where

Sf = slope of energy grade line. Sf is computed using the Manning's equation.

$$S_f = \frac{q^2.n^2}{h^{3.333}} \qquad (6)$$

Where

Δx = distance;

Z_u - Z_d = Δx . S_o;

h_u = 10.72 m;

h_d = 9.2 m;

q = 50 m^2/s;

n = 0.0175 ;and

S_o = 0.00025.

Equations (5) and (6) are solved to obtain Δx = 8934 m. Since we have an M2 profile and Δx< L the depth in the channel at the upstream end would be higher than 10.72 m. Therefore, the jump would form either in the stilling basin or on the glacis of the ogee weir.

As long as the tide level is between -3.39 m (-14.5+11.11) and -5.3 m, the water surface profile will be of M2 type and the jump will be stabilized in the stilling basin. If the tide level is > -3.4 m, there will be an M1 profile as shown in the figure. If the tide level is very high, the backwater effect may reach all the way up to stilling basin. In all these cases the jump would form either in the stilling basin or move to upstream locations.

Case (ii) Q = 60,000 m³/s

$Q = \frac{2}{3}C.\sqrt{2g}.L_e.{H_e}^{1.5}$ = 60,000 m^3/s

C for this case = 0.95 x 0.738=0.701

H_e = 6.47 m

TEL on the upstream side of the ogee weir = -3.5 + 6.47 = 2.97 m

TEL at the entrance to the stilling basin = TEL1= 2.97 – 0.5 = 2.47 m

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

Corresponding to the considered flow;

$$2.47 = -12.0 + y1 + \frac{34.1^2}{2.0.g.y1^2}$$

Where
y1 = pre-jump depth;
y1 = 2.20 m;
V1 = 15.5 m/s;
Froude Number = F1 = $\frac{V1}{\sqrt{g.y1}} = 3.34$
Sequent depth y2 = $y1 \times \left(\frac{\sqrt{1+8F1^2}-1}{2}\right) = 9.35\ m$

For the USACE energy dissipater,

Depth downstream of the dissipater = 0.85 x y2 = 7.922 m

Flow conditions in the channel are now checked:

S_o = 0.00025;
n = 0.0175;
q = 60,000/2200 = 27.27 m²/s

Critical depth = $h_c = \left(\frac{q^2}{g}\right)^{0.333} = 4.23\ m$

Normal depth = $h_n = \left(\frac{q.n}{\sqrt{S_0}}\right)^{0.6} = 7.72\ m$

It can be observed from the **Figure A2.320** that the water surface profile will be of M1 type.

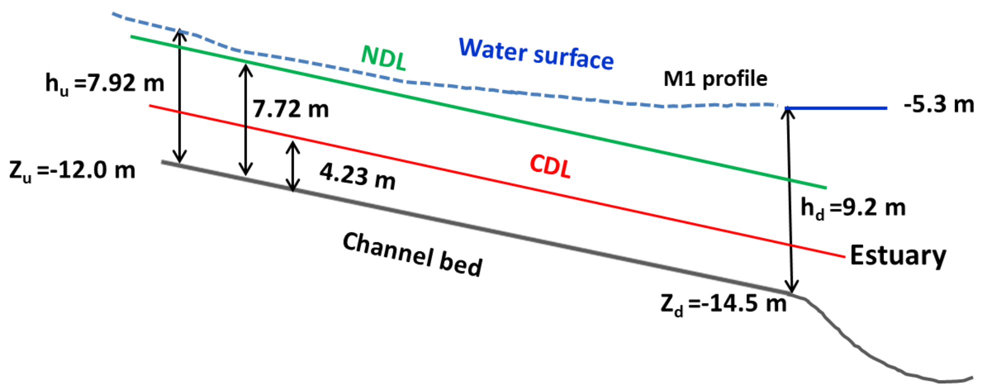


Figure A2.320: Water Surface Profile in the Spill Channel for Q = 60,000 m³/s

For a flow rate of q = 27.27 m²/s and downstream depth, h_d = 9.2 m, we determine at what distance upstream a depth h_u = 7.92 m would occur. For this purpose, we apply energy equation (Eq. 5) as earlier. For the given conditions, Δx = 17350 m. Since this is an M1 profile and Δx> L, the flow depth at the upstream end of the spill channel > 7.92 m. This indicates that the jump would form either in the stilling basin or on the glacis of the ogee weir.

Thus, for the provided stilling basin level =-12.0 m, the spill channel slope = 0.00025 and the roughness coefficient = 0.0175, the jump will form in the stilling basin.

(4) Arrangement in the energy dissipater

As per the guidelines of USACE, two rows of baffle blocks and an end sill are provided in the energy dissipater as shown in **Figure A2.318**. The spacing and dimensions of baffle blocks and end sill are worked out as follows, for a design flow = 110,000 m^3/s.

$F_1 < 4.6$

$L_1 = 1.5 \times y_2 = 18.9$ m **L_1 = 19.0 m**

$h = y_2/6 = 12.61/6 = 2.1$ m; **h = 2.1 m**

$L_2 = 2.5 \times h = 5.25$ m **L_2 = 5.3 m**

$h_s = h/2 = h/2 = 1.05$ m **hs = 1.1 m**

$L > L_1 + y_2 > 19 + 12.61 > 31.6$ m

$L > 4 \times 12.61 > 50.44$ m **L = 50 m**

Stagger the baffle blocks in rows 1 and 2.

Width of Baffle block = h = 2.1 m

Spacing between baffle blocks = 2.1 m

(5) Concrete apron and steel sheet piles

There is a concrete apron on which the ogee weir and the energy dissipater will be placed as shown in **Figure A2.321**.

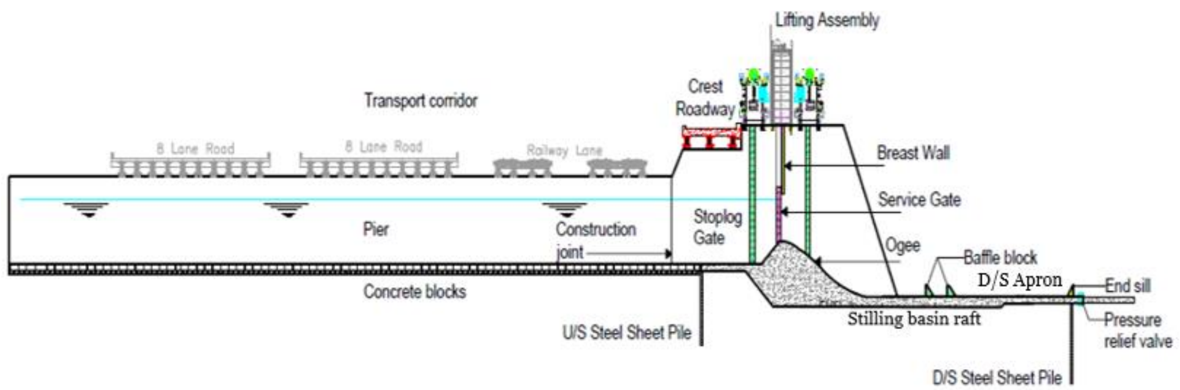


Figure A2.321: Schematic diagram of Flood Regulator with Concrete Apron and Sheet Piles

Typically, this concrete apron is designed based on the design concepts for hydraulic structures on permeable foundations i.e., hydraulic structures in rivers with alluvial soils. The apron 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 the apron 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.

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

Q = Maximum flood Discharge = 120,000 m^3/s (10% extra taken for factor of safety)

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 = $1.76\sqrt{m_r}$; m_r = average particle size in mm = 0.34 mm

f = 1.01

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

i)U/S Cutoff Size

As per the IS Code-6966; U/S cutoff should be equal to 1.25R. However, we are not applying a factor of safety because the channels on both upstream and downstream sides are going to be lined with concrete.

U/S Water level = +5.0 EL
U/S Floor Level = -7.00 EL
Scour Depth = 1R = 23.2 m
Bottom level of U/S Cutoff = +5.0-23.2 = -18.2 m
Depth of U/S Cutoff = 18.2 -7.0 = 11.2 m

Suggested size of U/S Sheet Pile = 11.5 m

ii) D/S Cutoff Size

As per the IS Code-6966; D/S cutoff should be equal to 1.5R. Once again, we are not applying this factor of safety.

D/S Water level = -12+10.72 = -1.3 m
D/S Floor Level = -12.0 m
Scour Depth = 1R = 23.2 m
Bottom level of D/S Cutoff = -1.3-23.2 = -24.5 m
Depth of D/S Cutoff = 24.5 -12 = 12.5 m

Suggested size of D/S Sheet Pile for cutoff = 12.5 m

(b) Length of the apron (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 **Figure A2.322**)

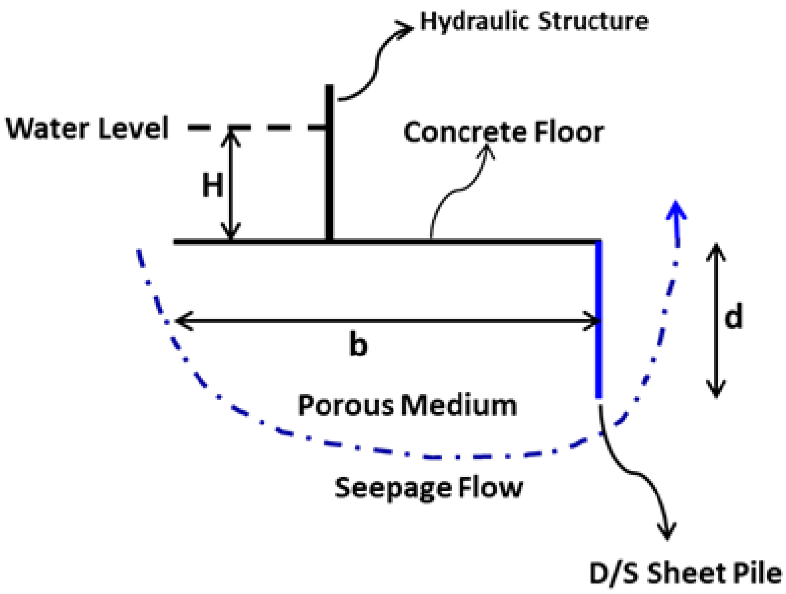


Figure A2.322: Schematic section for calculating Exit Gradient

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 = 12.5 m

Maximum Head causing the flow = MWL – Lowest Tail Water Level =+5.0 + 5.3 = 10.3 m

$\lambda = 3.4\alpha = 5.7; \quad b = 72\ m$

Provide a Floor Length of 76 m

Minimum Floor Length Required on D/S side = 50 m

Rest 26.0 m is provided on U/S of toe of the Ogee weir.

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

Two possible critical situations:

i) Water level is at +3.0 m (FRL); there is no flow over the spillway and the tail water level is at its minimum i.e., at -5.3 m. Head causing the flow = 8.3 m

ii) Water level is at +5.0 m (MFL); there is flow corresponding to PMF over the spillway and the tail water level is at -1.3 m. Head causing the flow = 6.3 m

Uplift calculations for KEY POINTS for Upstream piles

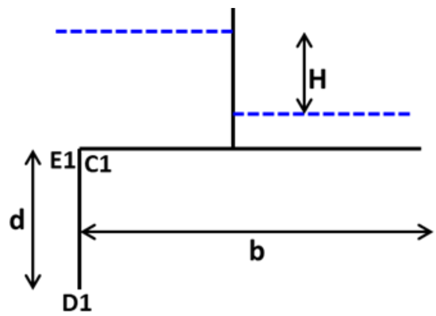


Figure A2.323: Schematic section for calculating Exit Gradient

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: $= 19\sqrt{\frac{(24.5-7.0)}{76.0}}.\frac{17.5+11.5}{76} = +3.5\%$

Correction for Thickness (Assume thickness = 1.0 m):

$$= \frac{76.5-65.9}{11.5} \times 1 = 0.9\%$$

$$\phi_{E1} = 100\%$$
$$\phi_{C1} = 65.9 + 3.5 + 0.9 = 70.3\%$$
$$\phi_{D1} = 76.5\%$$

Uplift calculations for KEY POINTS for D/S piles

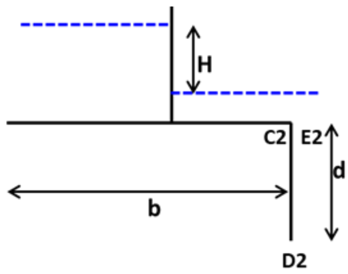


Figure A2.324: Schematic section for calculating Exit Gradient

Total length of floor = b = 76 m

Depth of D/S pile line below floor = d = 12.5 m

$$\alpha = \frac{b}{d} = 6.08$$

$$\phi_{C2} = 35.4\%$$
$$\phi_{D2} = 24.4\%$$
$$\phi_{E2} = 0\%$$

Correction for mutual interference: $= -19\sqrt{\frac{6.2}{76.0}}.\frac{6.2+12.5}{76} = -1.3\%$

Correction for Thickness (Assume thickness = 3.0 m)

$$= -\frac{35.4 - 24.4}{20} \times 3 = -2.6\%$$

$$\phi_{E2} = 0\%$$
$$\phi_{C2} = 35.4 - 1.3 - 2.6 = 32\%$$
$$\phi_{D2} = 24.4\%$$

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

Unbalanced pressure head at Key Points for critical case (a): (Figure A2.325)

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

$$h_{C1} = [(0.70 \times 8.3 + (-5.3)) - (-7.0)] - 10.0 = -2.49\ m$$

$$h_{C2} = [(0.32 \times 8.3 + (-5.3)) - (-12.0)] - 6.7 = 2.7\ m$$

$$h_{C3} = [-0.56 + 12] - 6.7 = 4.74\ m$$

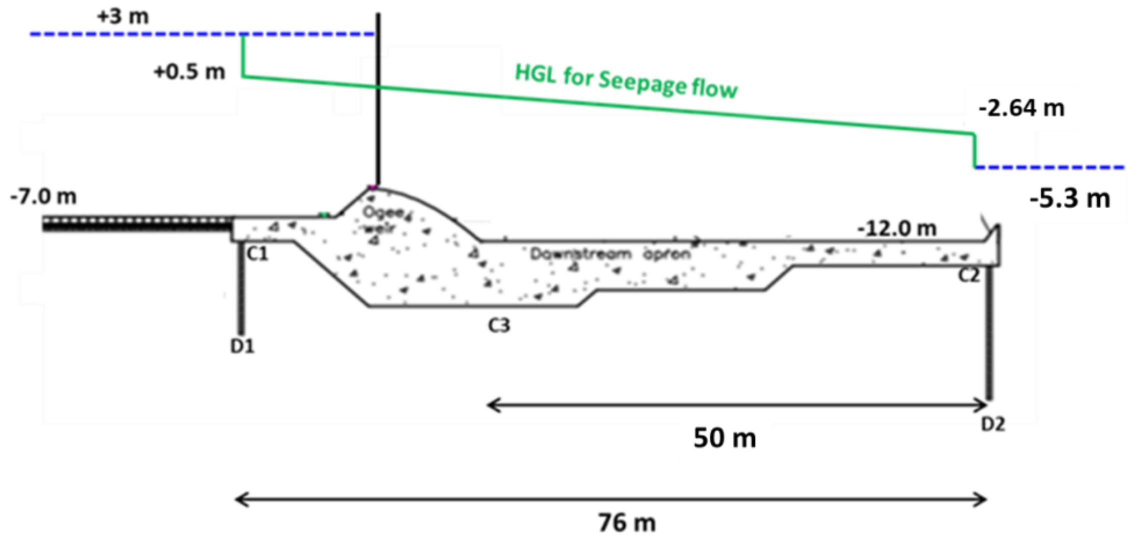


Figure A2.325: Schematic diagram showing Hydraulic Grade Line for Seepage: Case (a)

Unbalanced Pressure Head at Key Points for Critical Case (b): (Figure A2.326)

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

$$h_{C1} = \left[\left(0.70 \times 6.3 + (-1.3)\right) - (-7.0)\right] - 12.0 = \ -1.7\ m$$

$$h_{C2} = \left[\left(0.32 \times 6.3 + (-1.3)\right) - (-12.0)\right] - 10.7 = \ 2.0\ m$$

$$h_{C3} = [2.3 + 12] - 3.8 = \ 10.5\ m$$

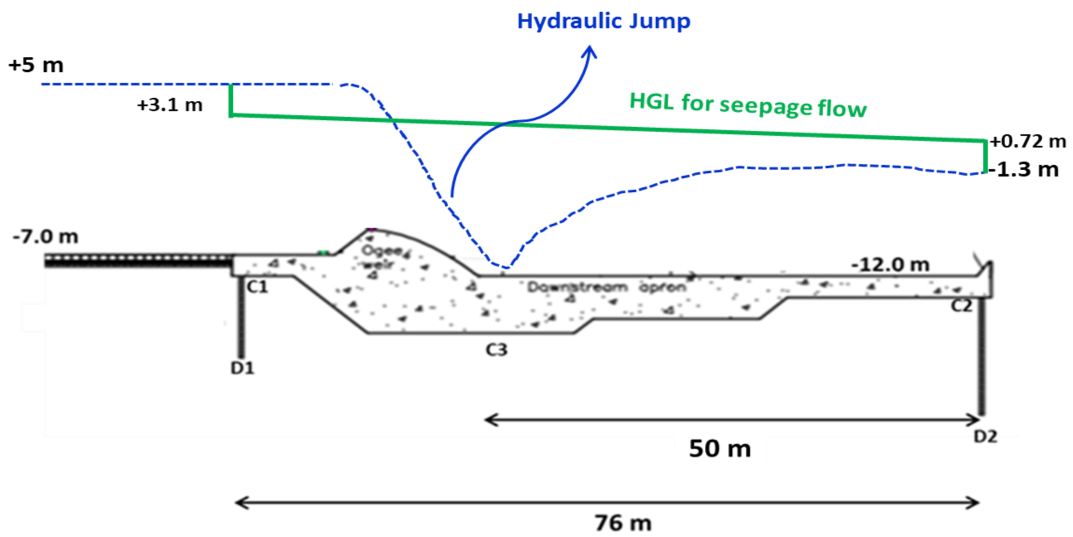


Figure A2.326: Schematic diagram showing of Hydraulic Grade Line for Seepage: Case (b)

<u>Critical values of unbalanced head</u>

At the toe of Ogee = 10.5 m
At the Downstream end of stilling basin = 2.7 m
The thickness of floor should vary from 10.5/1.4 = 7.5 m to 1.9 m to counter this unbalanced uplift pressure.

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.

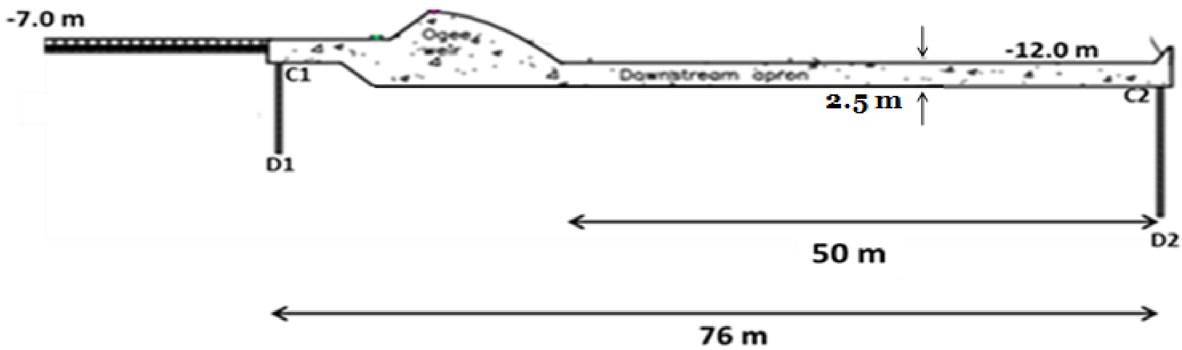


Figure A2.327: Schematic diagram showing thickness of stilling basin raft

2.3.7 Geotechnical Design

2.3.7 Geotechnical Design

The design is based on the geotechnical investigation carried along the spillway location. A total of 35 investigations combining 22 boreholes and ECPT (13 numbers) was 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 A2.303**. The engineering properties derived from CIRIA Report 143 for the corresponding boreholes are tabulated in **Table A2.304** and **A2.305**. A copy of the soil investigation report is given in Annex 1. The geotechnical interpretative report is given in Enclosure A.

2.3.7.1 Critical geotechnical strata

The components and the corresponding boreholes locations are shown in **Figure A2.328.**

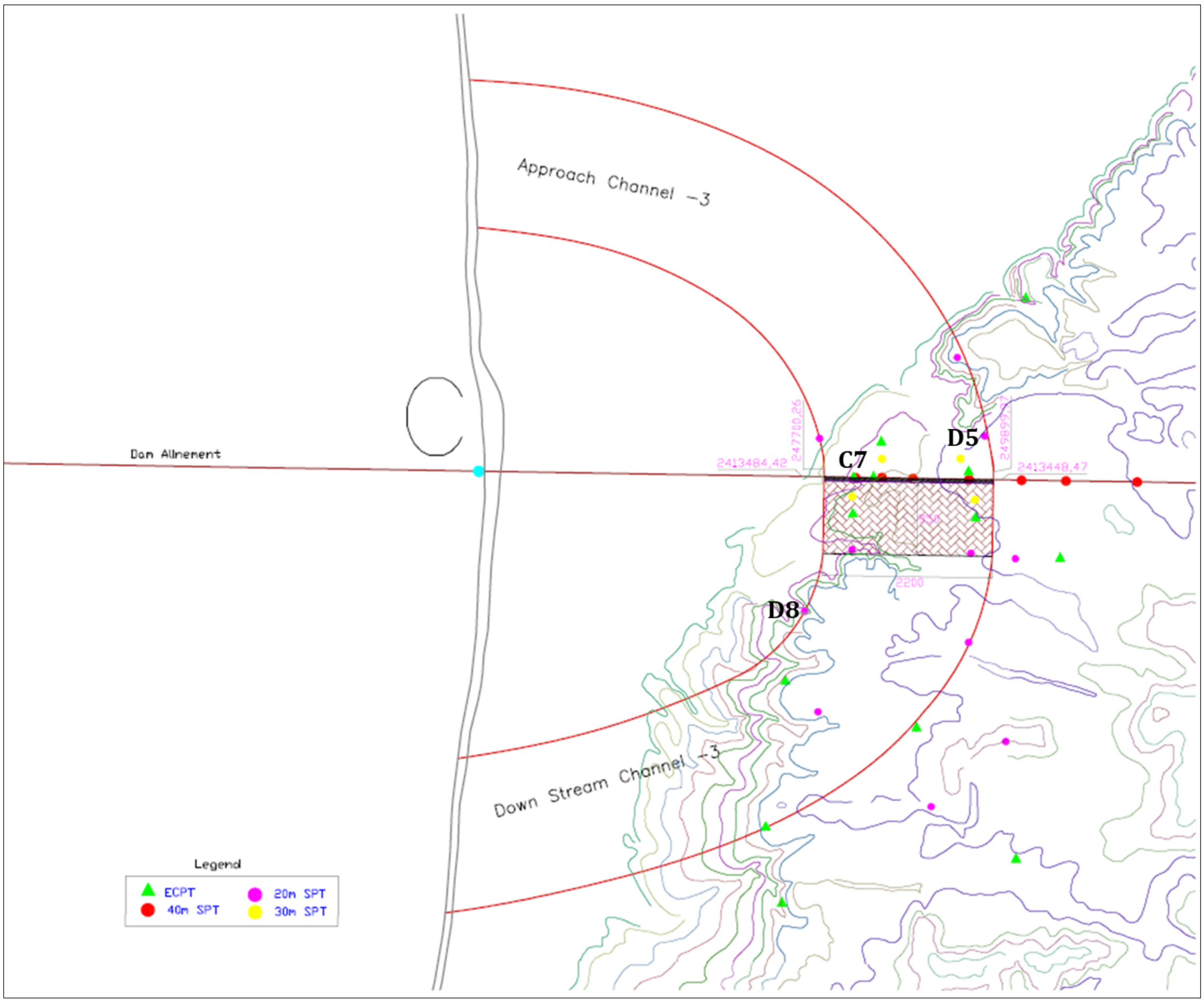


Figure A2.328: Boreholes and the critical boreholes adopted for Design

Table A2.303: Borehole ID under each component

S. No.	Location	Component	Borehole ID
1	Flood Regulator	Abutment	D5
2		Pier Wall and Ogee	C7

Table A2.304: Design of Cellular Coffer Dam: Shear Parameters used for design

Borehole C1			
Depth(m)	Strata	Shear Parameters	
		Cohesion (kPa)	Φ (degree)
0 – 1.5	Soft clay	4.5	-
1.5 – 7.5		9	-
7.5 – 13.5	Stiff clay	123	-
13.5 – 27		101	-
27 – 40	Dense sand	-	40

Borehole C2			
Depth(m)	Strata	Shear Parameters	
		Cohesion (kPa)	Φ (degree)
0 – 6	Soft clay	5	-
6 – 7.5		13.5	-
7.5 – 10.5	Silt	-	30
10.5 – 21	Stiff clay	98	-
21 – 25.5		114	-
25.5 – 31.5		107	-
31.5 – 33		171	-
33 – 36		240	-
36 – 37.5		302	-
37.5 – 40	Dense sand	-	40

Borehole C3			
Depth(m)	Strata	Shear Parameters	
		Cohesion (kPa)	Φ (degree)
0 – 1.5	Stiff clay	31.5	-
1.5 – 7.5	Soft clay	5	-
7.5 – 10.5	Stiff clay	64.5	-
10.5 – 13.5		63	-
13.5 – 16.5		121	-
16.5 – 30	Silt	-	30
30 – 40	Dense sand	-	40

Borehole C7			
Depth(m)	Strata	Shear Parameters	
		Cohesion(kPa)	Φ (degree)
0 – 4.5	Soft clay	5	-
4.5 – 10.5	Stiff clay	75	-
10.5 – 16.5		70	-
16.5 – 21		110	-
21 – 22.5		230	-
22.5 – 28.5		90	-
28.5 - 40	Dense sand	-	40

Table A2.305: Spill Channel Design: Shear Parameters used for design

Borehole D4			
Depth(m)	Strata	Shear Parameters	
		Cohesion (kPa)	Φ (degree)
0 – 6.5	Soft clay	0	-
6.5 – 9.5	Stiff clay	62	-
9.5 – 15.5		135	-
15.5 – 17		90	-
17 – 20		120	-

Borehole D8			
Depth(m)	Strata	Shear Parameters	
		Cohesion (kPa)	Φ (degree)
0 – 0.5	Soft clay	0	-
0.5 – 3.5		5	-
3.5 – 6.5		0	-
6.5 – 9.5		5	-
9.5 – 17.5	Stiff clay	95	-
17.5 – 20		140	-

2.3.7.2 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, settlement and pile foundation under stilling basin 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 2.3.6 (5). The schematic drawing showing the varying thickness of stilling basin is given below in **Figure A2.329.**

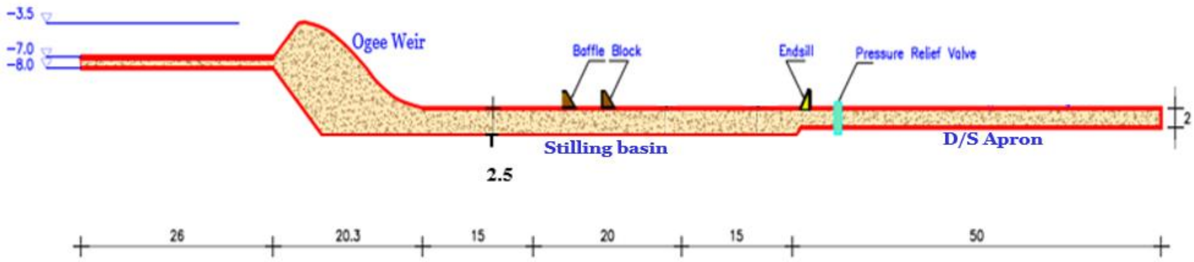


Figure A2.329: Schematic diagram showing thickness of stilling basin raft

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 A2.330** 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 A2.306.** Bearing capacity calculations for each section are given in Enclosure 1.

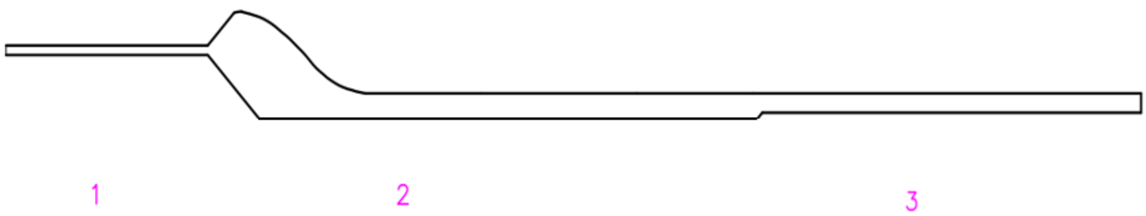


Figure A2.330: Different sections considered for SBC calculations

Table A2.306: Bearing Capacity for each Section

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

C) Settlement analysis

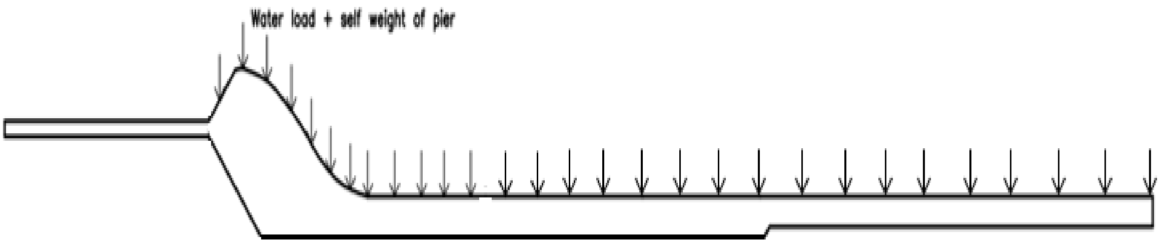


Figure A2.331: Typical Section of the stilling basin raft

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. Critical depth is considered as 15 x diameter of pile.
Recommended pile diameters and length are 1 m/ 1.5 m and 20 m respectively. **Table A2.307** shows the safe axial and uplift capacities for the same. **Figure A2.332** shows the typical cross section of pile foundation under stilling basin.

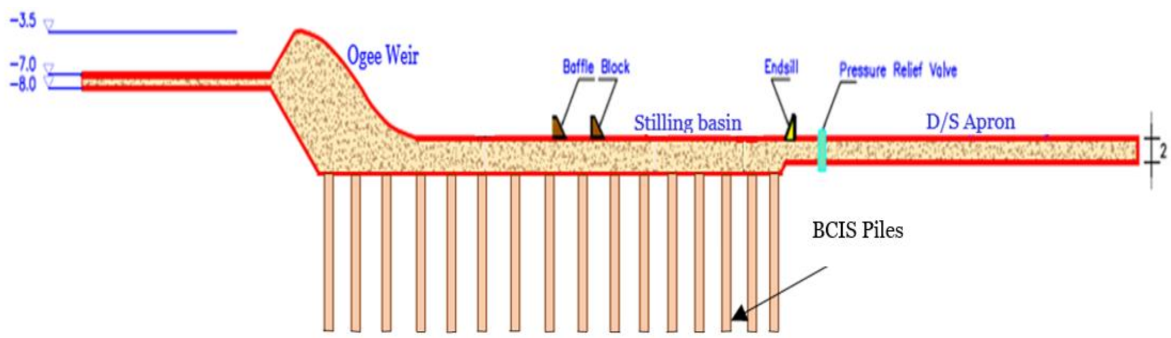


Figure A2.332: Schematic diagram showing pile foundation under stilling basin

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. **Table A2.308** and **Table A2.309** represents the number of piles and spacing for various diameters w.r.t different boreholes. Pile capacity calculations and number, spacing of piles required are given in Enclosure 2.

Table A2.307: Pile capacities for 1 m & 1.5 m diameters (20 m long)

BH	Diameter of pile (m)	Length of the pile (m)	Safe Axial capacity (t)	Uplift capacity (t)
C1	1	20	425	180
	1.5		990	290
C2	1		370	140
	1.5		925	225
C3	1		400	160
	1.5		950	260
C7	1		420	180
	1.5		985	290

Table A2.308: Number and spacing required for 1 m dia. and 20 m long piles

BH	Total number of piles required	Spacing between piles (m)
C1	8340	4.2
C2	11015	3.7
C3	9500	3.9
C7	8170	4.25

Table A2.309: Number and spacing required for 1.5 m dia. and 20 m long piles

BH	Total number of piles required	Spacing between piles (m)
C1	5225	5.3
C2	6786	4.7
C3	5890	5
C7	5280	5.3

2.3.7.3 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.

Static Earth Pressure					
Active Earth Pressure Coeff., Ka	0.2				
Due to surcharge pressure	4.8	kPa			
Earth pressure due to backfill	42.4	kPa			
Thrust due to surcharge (P4)	127.2	kN/m	@	13.3	m
Thrust due to backfill (P5)	561.8	kN/m	@	8.8	m

Seismic Earth Pressure					
kae	0.4				
kae(1-kv)	0.3				
kae(1+kv)	0.4				
kpe	3.6				
kpe(1-kv)	3				
kpe(1+kv)	4.3				
Kae(1+-kv)	0.3				
Earth pressure due to Surcharge	9	kPa			
Earth pressure due to Backfill	79.5	kPa			
Thrust due to surcharge	238.5	KN/m	@	13.3	m
Thrust due to backfill	1053.4	kN/m	@	8.8	m

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 analysed for stability: section just adjacent to the abutment and that to the guide bund.Flank walls on the upstream side is proposed as shown in **Figure A2.333** and on the downstream, it is proposed as shown in **Figure A2.334.** Both static and pseudo-static cases have been analysed to meet the safety criteria. The flank wall is analysed for construction and post-construction phases. The surcharge during construction and post-construction is assumed to be constant.

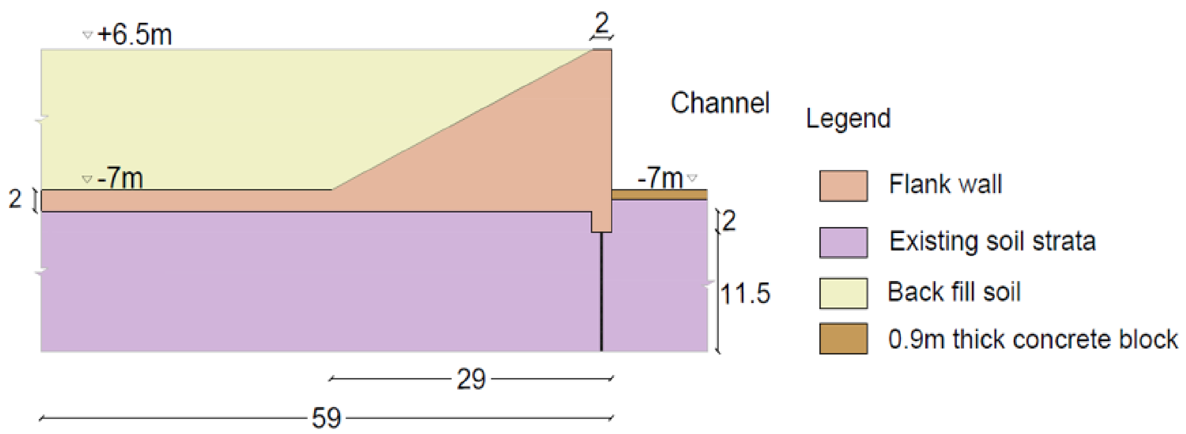


Figure A2.333: Cross Section of Upstream Vertical Flank Wall

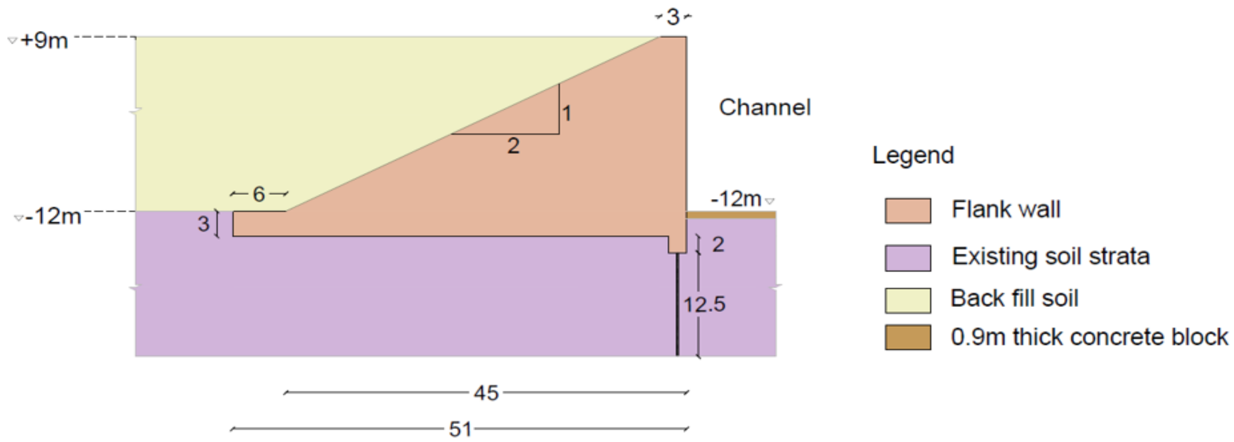


Figure A2.334: Cross Section of Downstream Vertical Flank Wall

i) Upstream Flankwall 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 utilised in the design as per IRC 6: 2014. The water level during construction and post construction is given below in **Table A2.310**

Table A2.310: Water level during different phase of construction

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

The analysis of the stability is as follows:

Top level of flank wall		=	6.5	m	w.r.t. MSL		
Level of slab top surface	-	=	7.0	m	w.r.t. MSL		
Height of soil to be retained	H	=	13.5	m			
Top Stem thickness	t	=	1.4	m			
Provide top stem thickness		=	2.0	m			
Bottom Stem length		=	29.0	m			
Thickness of slab	D	=	2.0	m			
Length of heel	L_h	=	30.0	m			
Length of toe projection	L_{tp}	=	0.0	m			
Length of slab	B	=	59.0	m			
Length of sheet pile projection	L_{sh}	=	2.0	m			
Surcharge	q	=	100.0	kN/sq. m			
water level in channel		=	-4.0	m			
Water depth in channel		=	3.0	m			
Water level @ backfill		=	5.0	m	w.r.t. MSL		
Water depth @ backfill		=	12.0	m			
friction angle	phi	=	30.0	deg			
Unit Weight		=	20.0	kN/m^3			
Submerged unit weight		=	10.0	kN/m^3			
Unit weight of water		=	10.1	kN/m^3			
Dead Weight of wall							
unit weight of concrete		=	25.0	Mpa			
W1		=	2950.0	kN/m	@	29.5	m
W2		=	675.0	kN/m	@	1.0	m
W3		=	4556.3	kN/m	@	11.0	m
W4 (sheetpile toe projection)		=	100.0	kN/m	@	0.0	m
W total (z bar)		=	**8281.3**	kN/m		16.6	m
Dead Weight of Soil above heel		=	8100.0	kN/m	@	44.0	m
Dead weight of soil inclined		=	3645.0	kN/m	@	20.0	
Dead Weight due to surcharge		=	5900.0	kN/m	@	29.5	m
		=	**17645.0**	kN/m			
Static Analysis:							
Earth pressure:							
H : V	1.0	:	2.0				
internal friction	del	=	0.3	20.0			
Inclination of inner wall with vertical	theta	=	0.5	26.6			
Angle of surcharge	beta	=	0.0				
K_a		=	1.0				
K_p		=	0.4				
Earth pressure due to soil		=	131.0	kPa			
Earth pressure due to surcharge		=	97.0	kPa			
thrust due to soil		=	883.9	kN/m	@	5.2	m
thrust due to surcharge		=	1309.5	kN/m	@	7.8	m
Uplift:		=	**2193.4**	**kN/m**			

Uplift pressure on heel side		=	30.2	kPa			
Uplift force		=	**4447.1**	kN/m	@	29.5	m
Hydrostatic load:		=					
Channel side		=	30.2	kPa			
channel side		=	**45.2**	kN/m	@	1.0	m
Backfill side		=	120.6	kPa			
backfill side water force		=	**723.6**	kN/m	@	4.0	m
Area of the water enclosing wall		=	504.0	Sq.m			
Weight of water (vertical force)		=	**5065.2**	kN/m	@	37.4	m
Check for sliding:							
Coefficient of friction		=	0.3				
Stabilising Force		=	9342.7	kN/m			
De-stabilising force		=	2917.0	kN/m			
Factor of Safety		=	3.2	safe			
Check for overturning							
Stabilising moment		=	930797.2	kNm/m			
De-stabilising moment		=	148800.1	kNm/m			
Factor of Safety		=	6.3	safe			
Length of sheet pile		=	11.5	m			
Depth of Sheet pile		=	13.5	m			
Passive pressure		=	50.0	kPa			
FORCE		=	337.2	kN/m			
50% PASSIVE		=	**168.6**	kN/m	@	9.0	m
Moment		=	**1517.2**	kNm/m			
Factor of safety sliding		=	3.3	safe			
Factor of safety overturning		=	6.2	safe			
Bearing Capacity Check							
Moment sum	M	=	781997.1	kNm/m			
Shear Force	V	=	26544.3	kN/m			
	X_{bar}	=	29.5				
Eccentricity	e	=	0.0	safe			
Minimum Pressure	P_{min}	=	448.1	kPa			
Maximum Pressure	P_{max}	=	451.7	kPa			
SBC		=	500.0	kPa			
Bearing Capacity Check			1.1	safe			

Seismic

Seismic Coefficients			
$k_a(1+kv)$	=	2.4	
$k_a(1-kv)$	=	1.7	
$k_p(1+kv)$	=	1.2	

$k_p(1\text{-}kv)$		=	0.9			
K_ae (1+-kv)		=	2.4			
K_pe (1+- Kv)		=	1.2			
Earth pressure due to soil		=	328.1	kPa		
Earth pressure due to surcharge		=	243.0	kPa		
Thrust due to soil		=	2214.3	kN/m	@	5.2 m
Thrust due to surcharge		=	3280.5	kN/m	@	7.8 m
Uplift		=				
Uplift pressure		=	30.2	kPa		
uplift pressure on heel side		=	135.7	kPa		
Uplift force		=	**4447.1**	kN/m	@	29.5 m
Hydrostatic load:						
Channel side		=	30.2	kPa		
		=	**45.2**	kN/m	@	1.0 m
Backfill side		=	120.6	kPa		
		=	**723.6**	kN/m	@	4.0 m
Area of the water enclosing wall		=	504.0	m^2		
Weight of water		=	**5065.2**	kN/m	@	37.4 m
		=				
Check for sliding:		=				
Coefficient of friction		=	0.3			
Stabilising Force		=	9342.7	kN/m		
De-stabilising force		=	6218.4	kN/m		
Factor of Safety		=	1.50	Safe		
Addl. Resisting force required		=	-15.0	kN/m		
		=				
Check for overturning		=				
Stabilising moment		=	930797.2	kNm/m		
De-stabilising moment		=	170949.2	kNm/m		
Factor of Safety		=	5.4	safe		
		=				
Length of sheet pile		=	11.5	m		
Depth of Sheet pile		=	13.5	m		
Passive pressure		=	162.0	kN/m		
Force		=	1093.5	kN/m		
50% Passive		=	**546.8**	kN/m	@	9.0 m
Moment		=	**4920.8**	kNm/m		
		=				
Factor of safety sliding		=	1.6	safe		
Factor of safety overturning		=	5.4	safe		
		=				
Bearing Capacity Check		=				
Moment sum	M	=	759848.0	kNm/m		
Shear Force	V	=	26544.3	kNm/m		
	X_{bar}	=	28.6			

Eccentricity	e	=	0.9	safe
Minimum Pressure	P_{min}	=	409.9	kPa
Maximum Pressure	P_{max}	=	489.9	kPa
SBC		=	500.0	kPa
		=	1.0	safe

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 utilised 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 below:

Phase	Highest Level w.r.t. MSL	Lowest Level w.r.t. MSL
Construction Phase	+5.2 m	-4.8 m
Post-construction Phase	+8.1 m	-5.3 m

The analysis of the stability is as follows:

Top of flank wall		=	9.0	m			
Base level of flank (top surface of downstream)		=	- 12.0	m			
Height of soil to be retained	H	=	21.0	m			
top stem thickness	t	=	2.1	m			
provide Top Stem thickness		=	3.0	m			
Bottom Stem Thickness		=	45.0	m			
Thickness of slab	D	=	3.0	m			
Length of heel	L_h	=	6.0	m			
Length of toe	L_t	=	0.0	m			
Length of slab	B	=	51.0	m			
Length of toe projection for sheet pile	L_{sh}	=	2.0	m			
Surcharge	q	=	30.0	kN/m			
H: V	1	:	2				
Highest possible water level (channel)		=	-4.8	m			
Water depth in channel(H3)		=	7.2	m			
Backfill water level		=	5.2	m			
Water depth @ backfill (H2)		=	17.2	m			
friction angle	phi	=	30.0	deg			
Unit Weight		=	20.0	kN/m^3			
Submerged unit weight		=	10.0	kN/m^3			
Unit weight of water		=	10.1	kN/m^3			
Dead Weight of wall							
unit weight of concrete		=	25.0	kN/m^3			
W1		=	3825.0	kN/m	@	25.5	m
W2		=	1575.0	kN/m	@	1.5	m
W3		=	11025.0	kN/m	@	17.0	m
W4 (sheet pile toe projection)		=	100.0	kN/m	@	1.0	m

Z bar		=	16525.0	m		17.4 m
Dead Weight of Soil free		=	2520.0	kN/m	@	48 m
Dead weight of soil inclined		=	8820.0	kN/m	@	31.0 m
Dead Weight due to surcharge		=	1530.0	kN/m	@	25.5 m
		=	12870.0	kN/m		
Static Case						
Earth pressure:						
internal friction	del	=	20.0	deg		
Inclination of inner wall with vertical	theta	=	26.6	deg		
Angle of surcharge	beta	=	0.0	deg		
K_a		=	1.0			
K_p		=	0.4			
Earth pressure due to soil		=	203.7	kPa		
Earth pressure due to surcharge		=	29.1	kPa		
thrust due to soil		=	2138.9	kN/m	@	7.0 m
thrust due to surcharge		=	611.1	kN/m	@	10.5 m
Uplift pressure		=	72.4	kPa/m		
uplift pressure on heel side		=	172.9	kPa/m		
Uplift force		=	6253.1	kN/m	@	25.5 m
Hydrostatic load:						
Channel side		=	72.4	kPa		
force		=	260.5	kN/m	@	2.4 m
Backfill side		=	172.9	kPa		
force		=	1486.6	kN/m	@	5.7 m
Area of the water enclosing wall		=	399.0	sq.m		
Weight of water		=	4010.4	kN/m	@	37.3 m
Check for sliding:						
Coefficient of friction		=	0.3			
Stabilising Force		=	10282.1	kN/m		
De-stabilising force		=	4236.5	kN/m		
Factor of Safety		=	2.4	safe		
Addl. Resisting force required		=	-2868.1	kN/m		
Check for overturning		=				
Stabilising moment		=	870929.9	kNm/m		
De-stabilising moment		=	189366.0	kNm/m		
Factor of Safety		=	4.6	Safe		
Addl. Moment reqrd.		=	-492198.0	kNm/m		
Length of sheet pile		=	12.5	m		
Depth of Sheet pile		=	14.5	m		
Passive pressure		=	53.7	kPa		
force		=	389.0	kN/m		

50% force		=	194.5	kN/m	@	9.7	m
Moment		=	1880.0	kNm/m			
Factor of safety sliding		=	2.5	safe			
Factor of safety overturning		=	4.6	safe			
Bearing Capacity Check							
Moment sum	M	=	702952.5	kNm/m			
Shear Force	V	=	27152.2	kN/m			
	X_{bar}	=	25.9	m			
Eccentricity	e	=	-0.4	safe			
Minimum Pressure	P_{min}	=	556.8	kPa			
Maximum Pressure	P_{max}	=	508.0	kPa			
SBC		=	2600.0	kPa			
Addl. BC needed		=	-2043.2	kPa	/m		
Seismic Analysis							
$k_ae(1+kv)$		=	2.4				
$k_ae(1-kv)$		=	1.7				
$k_pe(1+kv)$		=	1.2				
$k_pe(1-kv)$		=	0.9				
K_ae (1+-kv)		=	1.7				
K_pe (1+- Kv)		=	0.9				
Earth pressure due to soil		=	359.1	kPa			
Earth pressure due to surcharge		=	51.3	kPa			
thrust due to soil		=	3770.6	kN/m	@	7.0	m
thrust due to surcharge		=	1077.3	kN/m	@	10.5	m
Uplift pressure		=	72.4	kPa			
uplift pressure on heel side		=	172.9	kPa			
Uplift force		=	6253.1	kPa	@	25.5	m
Hydrostatic load:							
Channel side		=	72.4	kPa			
		=	260.5	kPa	@	2.4	m
Backfill side		=	172.9	kPa			
		=	1486.6	kPa	@	5.7	m
Area of the water enclosing wall		=	399.0	Sq.m			
Weight of water		=	4010.4	kN/m	@	37.3	m
Check for sliding:		=					
Coefficient of friction		=	0.3				
Stabilising Force		=	10282.1	kPa			
De-stabilising force		=	6334.4	kPa			
Factor of Safety		=	1.6	safe			
Check for overturning		=					

Stabilising moment		=	870929.9	kNm/m			
De-stabilising moment		=	205683.0	kN/m			
Factor of Safety		=	4.2	safe			
Length of sheet pile		=	12.5	m			
Depth of Sheet pile		=	14.5	m			
Passive pressure		=	123.3	kPa	@	9.7	m
Force		=	893.6	kN/m			
50% Passive		=	446.8	kN/m			
Moment		=	4318.9	kNm/m			
Factor of safety sliding		=	1.7	safe			
Factor of safety overturning		=	4.2	safe			
Bearing Capacity Check							
Moment	M	=	665247.0	kNm			
Shear Force	V	=	27152.2	kN/m			
	X_{bar}	=	24.5				
Eccentricity	e	=	1.0	safe			
Minimum Pressure	P_{min}	=	469.8	kPa			
Maximum Pressure	P_{max}	=	595.0	kPa			

(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.

(e) 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 bank slope of 1:2.5, the thickness of mattress required will be equal to or more than 0.45 m as per IRC: SP:116-2018. On a conservative side, revet mattress of 0.5 m thick is proposed on bed and slopes of the channels. The maximum wave heights that the spillway can experience both on the upstream and downstream side are 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 of 0.25 m thick are to be provided. The mattress shall be sub-divided into 2 compartments by the insertion of diaphragm made of the same mesh as the rest of the mattress. The diaphragms shall be secured in proper position at the base with a continuous spiral wire, in such a manner that no additional tying at junction will be necessary.

The proposed revet mattress consists of mechanically woven double twisted hexagonal shaped wire mesh and a non-woven geotextile 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 opening. Each size may allow a variation of 5% oversize or 5% undersize, or both. Figure xx shows a typical revet mattress.

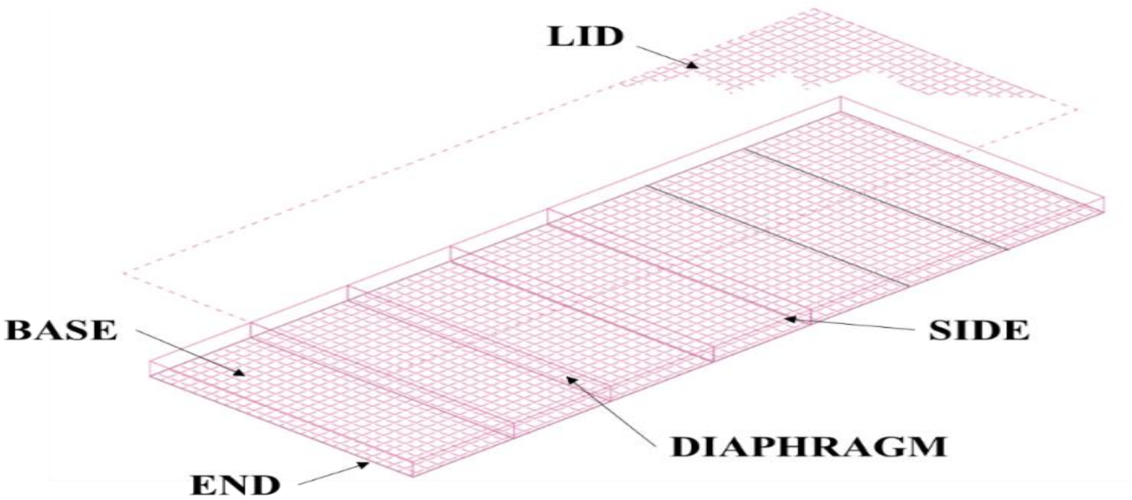


Figure A2.335: Revet Mattress Configuration

2.3.7.4 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 A2.336** shows the plan of circular cellular cofferdams around the flood regulator.

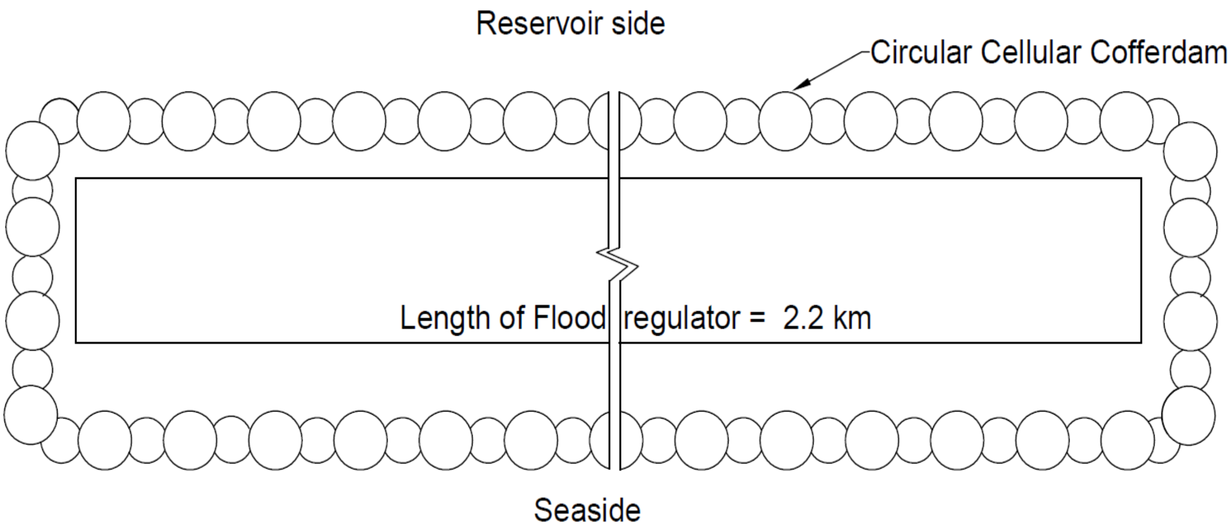


Figure A2.336: Plan showing cofferdam around flood regulator

b) Design of Circular Cellular Cofferdam

Soil Profile

Cohesion values are interpreted from CIRIA which correlates the plasticity index to Cu/N where N is the SPT value. The high tide level in the area is + 6.2 m w.r.t MSL. **Figure A2.337** shows the soil profile.

Dimensions of Cofferdam

From IS code, cellular structures have been checked against cell shear, sliding, overturning (in case of clays). Taking the diameter of cofferdam, as 23.95 m, the factor of safety values for various mechanisms is greater than the required values. Design of cofferdam calculations and check for stability against various parameters are given in Enclosure 4.

From IS: 9527 part 4, the dimensions of each cell for the corresponding diameter of 23.95m are given in **Table A2.312**. The plan of cellular cofferdam showing all dimensions is given in **Figure A2.337.**

Sheet pile Section

For the construction of circular cellular cofferdams, straight web steel sheet piles confirming to ISPS 100F of IS: 2314-1963 shall be used. The material of sheet piles should contain 0.2 to 0.35% of copper for imparting corrosion resistance. The typical plan of piling section is given in **Figure A2.338**.

Table A2.311: Factor of safety for various parameters

Mechanism	Required FOS	Obtained FOS
Sliding	1.25	2.97
Overturning	2	2

Table A2.312: 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

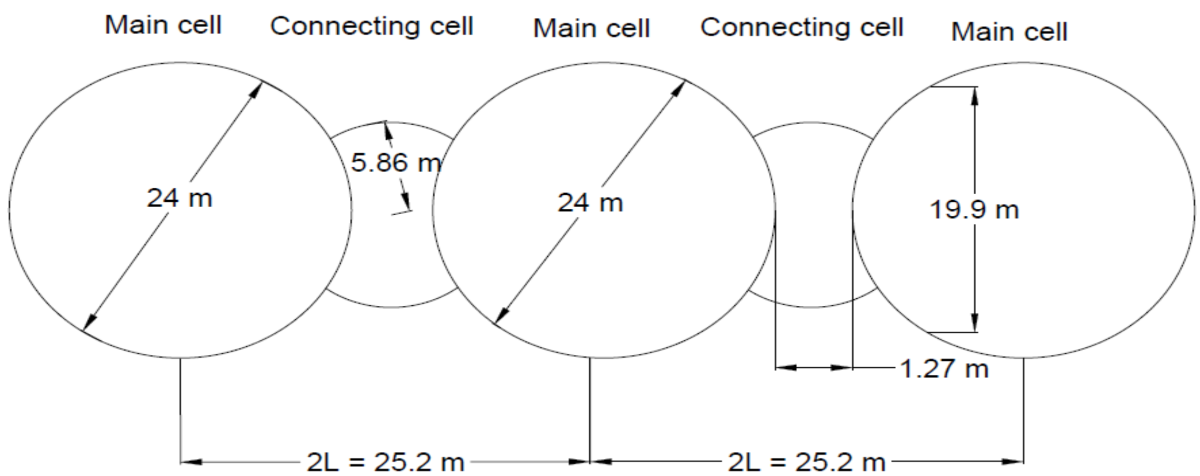
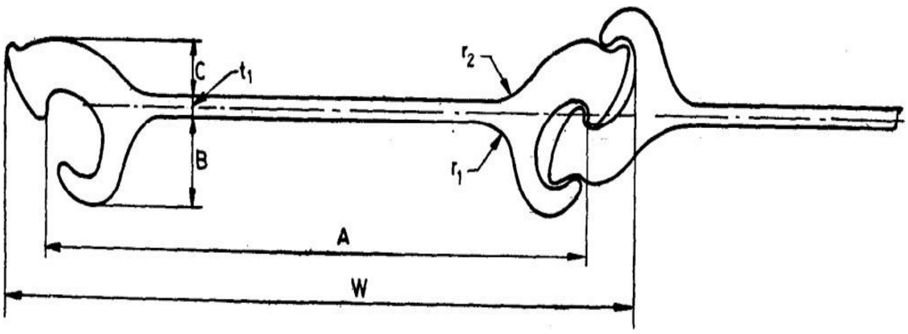


Figure A2.337: Plan of Cellular Cofferdam



W = 445 mm

A = 400 mm

B = 40 mm

C = 25 mm

t_1 = 9 mm

r_1 = 13 mm

r_2 = 18 mm

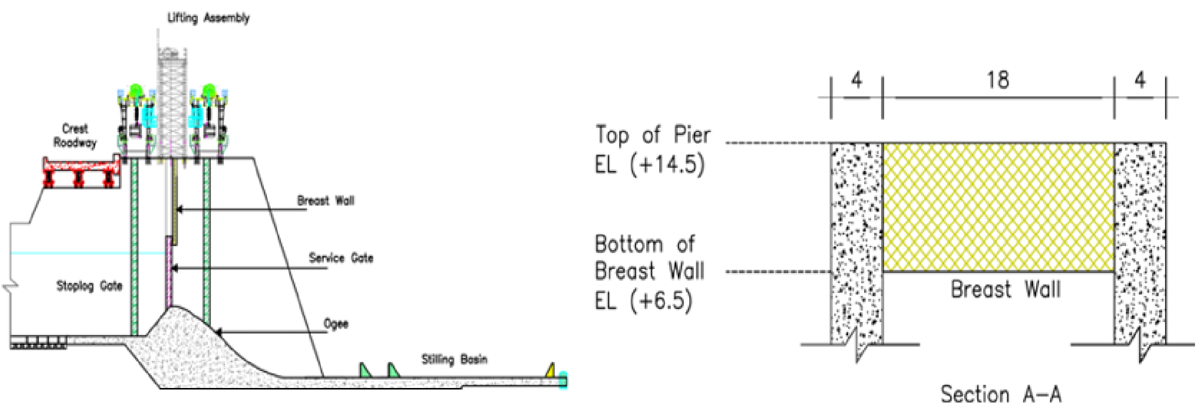
Figure A2.338. Typical Flat type Piling Section

2.3.8 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.

(1) Breast wall

The top level of gate is estimated to be very high and a single unit of vertical lifting gate is not possible over 20m height. So, it was decided to provide 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 two parts are assumed to be constructed monolithic or separated by a joint and provided with a seal in between. 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 would be designed for bending in both the horizontal and vertical directions. The loads considered in analysis are listed under loads and load combinations. Since the vertical stem is fixed to the piers and abutments, full weight of the stem may not transfer to the beam. A fair proportion at the discretion of the designer may be taken as the transferred load. When the stem and beam are monolithic, the breast wall has to be checked for torsion also caused by the water trust. The area of the second stage concrete for the embedded parts should be neglected while analysing the beams. The configuration of breast wall is shown in **Figure A2.339**.



FigureA2.339: Configuration of Breast Wall

A) Codal References

a) IS 456-2000: Plain and Reinforced Concrete - Code of Practice
b) IS 4651-2020 (Part-4): Code of practice for planning and design of ports and harbours
c) IS 11130-1984: Criteria for Structural Design of Barrages and Weirs
d) IS 875-1987 (Part-1): Code of Practice for Design Loads (Other Than Earthquake) For Buildings and Structures.
e) IS 875-1987 (Part 3): Code of Practice for Design Loads (Other Than Earthquake) For Buildings and Structures
f) IS 1893-1984: Criteria for Earthquake Resistant Design of Structures

B) Loads

The beam of the breast wall spanning between the pier and abutment/pier shall be designed to resist moments due to

a) Dead load of the breast wall;
b) Uplift;
c) Water pressure;
d) Seismic forces and moments;
e) Hydrodynamic forces due to seismic conditions and
f) Wind load.

The beam and stem are designed to resist the moments in both horizontal and vertical directions. The beam has been checked for torsional moments also and suitably reinforced. The stem may also be checked as a deep beam subjected to the various loads and moments though generally it may not govern the requirements of reinforcement.

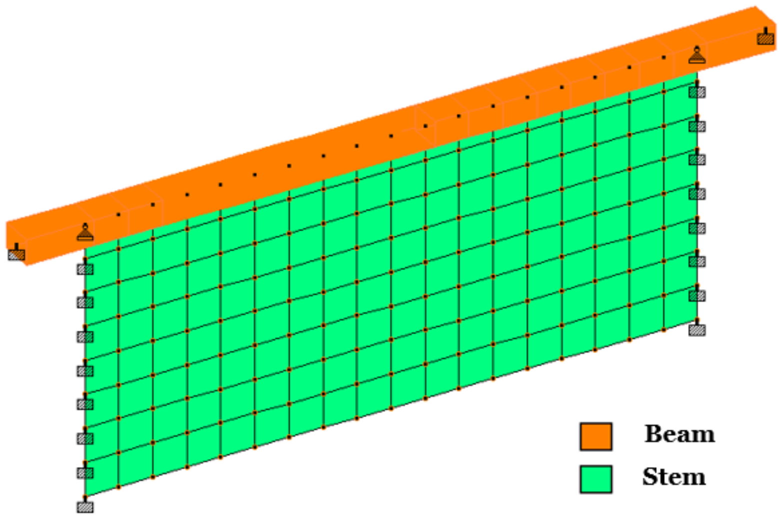
C) Loads Combinations:

Table A2.313. Load combinations considered for Design

Cases \ Loads	Dead Load (DL)	Live Load (LL)	Hydro Static Force (H_SF)	Hydro Dynamic Force (H_dF)	Wind Load (W_iL)	Wave Load ($W_{av}L$)	Seismic Load (SL)
Normal Condition	1.5	1.5	1.0	-	1.0	1.2	-
Seismic Condition	1.5	1.5	-	-	-	-	-
	1.2	1.2	-	1.2	-	-	1.2
	1.5	-	-	1.5	-	-	1.5
Extreme Condition	1.2	1.2	1.0	1.2	-	1.0	1.2
Temporary Condition	1.2	1.2	1.0	-	-	1.0	-
Reversal Load Condition	0.9	0.9	1.0	-	1.5	1.0	-
	0.9	0.9	1.0	1.5	-	1.0	1.5

D) Design Methodology:

The breast wall parts namely, vertical stem and horizontal beam are modelled and analysed in STAAD pro, which is a Finite Element Analysis software. **Figure A2.340** shows the 3D view of Breast wall.



FigureA2.340: 3D STAAD model view of Breast Wall

E) Load Calculation:

Dead Load:

The dead load or the self-weight of this structure is calculated by its unit weight of materials used as per IS875 (part1)-1987- Reaffirmed (2008).

Hydro-static Force:

Water level (on Sea side)	=	+ 8.1 m
Water pressure (2.3 x 10)	=	23 kN/m2
Maximum height of water	=	2.3 m

Hydro-dynamic Force:

Hydrodynamic pressure, P (kN/m2) = Cs x αh x w x h = 2.15 kN/m2

Cs = coefficient which varies with shape and depth

αh = design horizontal seismic coefficient

w = unit weight of water in kN/m^2, and

h = depth of reservoir in m.

Wind Load:

Design Wind Pressure, F_T = P_z x G x A x C_D

Where,

P_z is the hourly mean wind pressure in N/m2 (calculated as per IS-875-Part-3)

Wind speed	=	50 m/s
Risk coefficient factor	=	1.08
k_2 Factor	=	1.09
Topography factor	=	1
Cyclone importance	=	1.15
Design wind speed (Vb x k1 x k2 x k3 x k4)	=	67.69 m/s
Design wind speed (0.6 x V_z^2)	=	2.75 kN/m^2

Wave Load:

As per IS 4651, Annexure B-1,

Depth of water	=	18.5	m
Height of free wave	=	2	m
Weight of water	=	10	kN
Time Period	=	12	s
Deep water wave length, Lo	=	224.64	m
d/Lo	=	0.060	
d/L (As per wave table)	=	0.1403	
L	=	96.22	
($cosh^2$ x pi x d)/L	=	0.635	

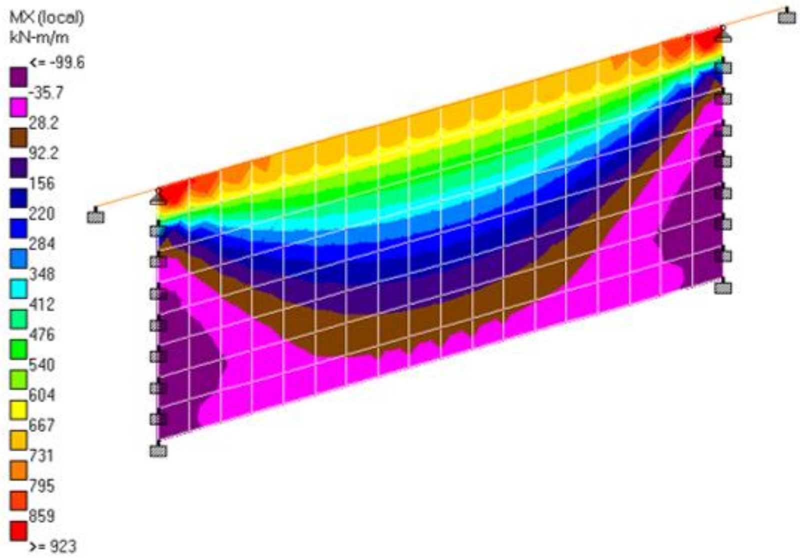
Earthquake loads:

Earthquake loading as applied in the structure by using seismic definition in Staad Pro Software relying to the standard IS 1893 Part 1 -2016 in seismic coefficient method. Currently, the India Seismic Code IS 1893 (Part 1) is under revision. As per the new standard the location of the dyke falls in Zone IV against Zone III in existing standard. The safety assessment level taken for dyke (0.36g) is considered for structural design for flood regulator (corresponding to a return period of 975 years). The acceleration corresponding to the extreme event is taken two levels higher as 0.54g (corresponding to a return period of 4995 years). Also, the return period for the serviceability check is taken one level lower as 475 years; the associated acceleration value is 0.30g.

F) Design Forces and Reinforcement Calculation:

Design of Beam

Beam of size 550mm x 800mm (B x D) is modelled monolithically with the vertical stem. It is designed in such a way to carry the torsional moments generated by the hydrostatic forces on the stem. Beam of 22m (c/c) span is considered to be supported between 4m thick wall piers. Beam is designed as deep beam considering the above-mentioned loads and load combinations. Analysis results of breast wall are shown in **Figure A2.341**. To compensate the design moments 0.92% of tension reinforcement at sagging face (mid-span) and 0.5% of compression reinforcement in hogging face (near supports) are provided. 0.1% of side face reinforcement is provided since it is designed as deep beam. Shear stress is found to be within permissible limit as per IS-456-2000.



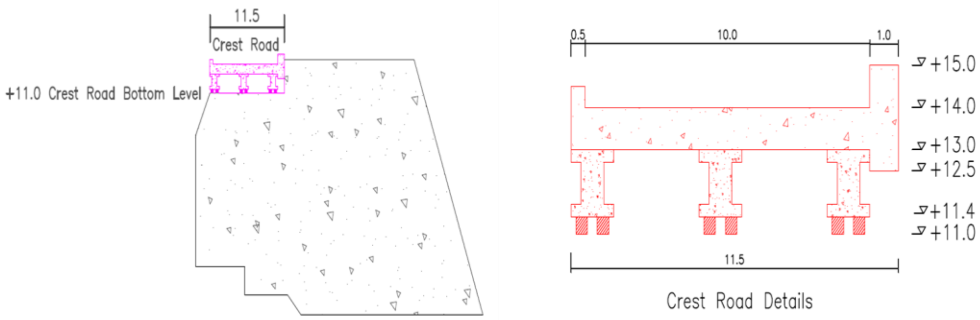
FigureA2.341: Analysis Results of Breast Wall

2) Crest Road

The configuration of Crest Road on piers is shown in **Figure A2.342**

A) Design Criteria:

Crest road is an inspection road meant for inspecting and operating gantry girders and gates. Crest road is designed as a deck slab supported by I-girders placed on the wall piers. I-girders are spanned between pier to abutment/pier.



FigureA2.342: Configuration of Crest Road

B) Codal References:

a) IS 456-2000: Plain and Reinforced Concrete - Code of Practice
b) IRC 6 - Standard Specifications and Code of Practice for Road Bridges.

C) Loads:

Loads considered in design of crest road includes

a) Dead load;
b) Super-imposed dead load;
c) Temperature & Wrapping stresses;
d) Vehicular load;
e) Wind load; and
f) Earthquake load.

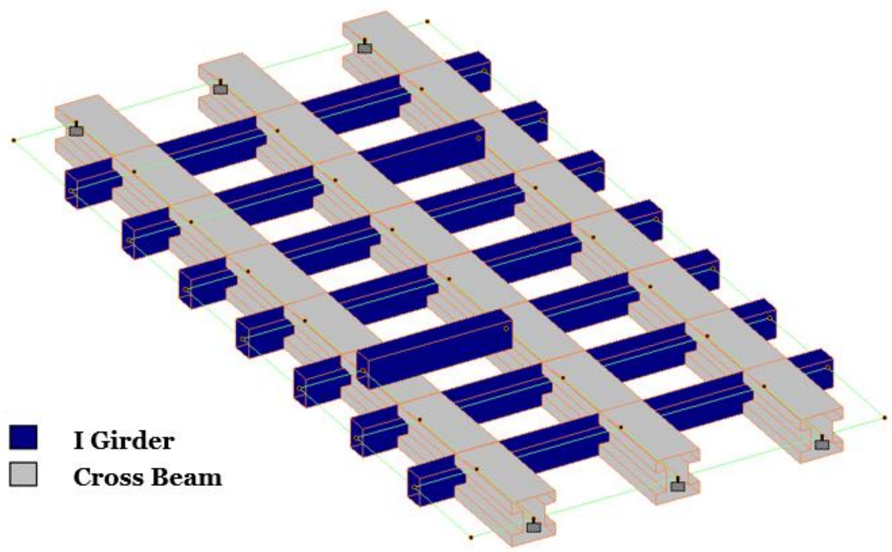
D) Loads Combinations:

Table A2.314. Load combinations considered for Design

DESCRIPTION	DL	SIDL	TS	VL	WL(X)	WL(Z)	EQ(X)	EQ(Z)
Basic combination – ULS design	1.35	1.35		1	1.5	1.5	-	-
Seismic combination ULS design	1.35	1.35		1	-	-	1.5	1.5
Service Rare combination	1	1	1	1	1	1	-	-
Service frequent combination	1	1	0.6	1	0.6	0.6	-	-
Service Quasi permanent combination	1	1	-	1	-	-	-	-

E) Design Methodology:

The Crest Road is designed as a deck slab supported by girders. Girders are designed as 'I'-beam supported on wall piers. The reinforcement details of beams in flange portion is governed by bending moments in the beam whereas the shear reinforcements are provided in web part of the girder. All the components are modelled and analysed in STAAD pro software. 3D model of Crest Road is shown in **Figure A2.343.**



FigureA2.343: 3D STAAD view of Crest Road

F) Load Calculation:

Dead Load:

The dead load or the self-weight of this structure is calculated by its unit weight of materials used as per IS875 (part1)-1987- Reaffirmed (2008).

Super-Imposed Dead Load:

The super imposed load carried out by girder or member shall consists of the weight of the Crash Barrier and the Wearing Coat. A uniform carriage way load of 3 kN/m^2 is applied.

Vehicular Load:

Crest road is used for transporting gates, gantry girders for operation and maintenance of spillway gates. Considering special crane loading and Heavy vehicular loadings CLASS 70 R loading as per clause 204.1 of IRC 6 2010 is used. The Loading diagram of Class 70R is given below.

Table A2.315: Two Class 70 R Loadin

Class 70 R - Loading			
Axle Load	Wheel load	Distance	Impact factor
80	40	-	1.21
120	60	4	1.21
120	60	1.5	1.21
170	85	2.3	1.21
170	85	1.35	1.21
170	85	1.35	1.21
170	85	3	1.21

Wind Load:

Design Wind Pressure, F_T	=	P_z x G x A x C_D
where, P_z is the hourly mean wind pressure in N/m2 (calculated as per IS-875-Part-3)		
Wind speed	=	50 m/s
Risk coefficient factor, k1	=	1.08
k_2 Factor	=	1.09
Topography factor, k_3	=	1
Cyclone importance, k_4	=	1.15
Design wind speed (Vb x k1 x k2 x k3 x k4)	=	67.69 m/s
Design wind speed (0.6 x V_z^2)	=	2.75 kN/m2

Temperature & Wrapping Stresses:

σf (WLf/(2*10^-4))	=	0.5 kN/m2
where, W, unit weight of concrete	=	24 kN/m3
f, Coefficient of subgrade friction	=	1.5

Earthquake loads:

Earthquake loading as applied in the structure by using seismic definition in Staad Pro Software relying to the standard IS 1893 Part 1 -2016 in seismic coefficient method. Currently, the India Seismic Code IS 1893 (Part 1) is under revision. As per the new standard the location of the dyke falls in Zone IV against Zone III in existing standard. The safety assessment level taken for dyke (0.36g) is considered for structural design for flood regulator (corresponding to a return period of 975 years). The acceleration corresponding to the extreme event is taken two levels higher as 0.54g (corresponding to a return period of 4995 years). Also, the return period for the serviceability check is taken one level lower as 475 years; the associated acceleration value is 0.30g.

G) Design Forces and Reinforcement Calculation:

Analysis results of Crest Road is shown in **Figure A2.344**

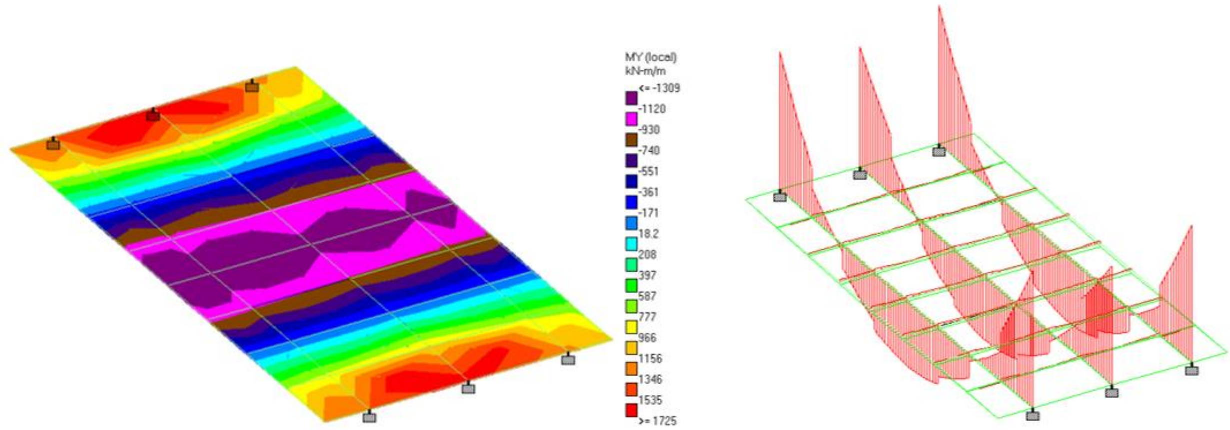


Figure A2.344: Analysis Results of Crest Road

a) Design of Deck slab

Crest road is modelled as a deck slab of 1m thick and analysed for the above-mentioned loads and load combinations. Bending moments are extracted from STAAD pro in both horizontal and vertical directions. Reinforcement of 0.55% is provided on the transverse direction of roadway and 0.25% is provided on the longitudinal direction of roadway. Shear stress is found to be within permissible limits as per IS-456-2000.

b) Design of I- Beam:

As per the computed moments from STAAD pro, I-girder of flange width 1200mm and 300mm thick (both top and bottom) with web width of 600mm and thickness of 800mm is found to be sufficient. Flange reinforcement of 0.25% and Web reinforcement of 0.5% is provided. The girder is checked for serviceability condition for Deflections due to loads, shrinkage and creep. And the deflection is found to be within permissible limits.

c) Design of Cross Beam

Cross beam of 600mm x 800mm (B x D) is provided between two girders at every 2.75m spacing in longitudinal direction of road way. 0.3% of tension reinforcement is provided at the bottom part of the beam and 0.2% of compression reinforcement is provided at the top part of the beam. Since the depth of beam exceeds 750mm 0.1% of side face reinforcement is provided. Shear stress is found to be within permissible limits as per IS-456-2000.

3) Ogee Weir

A) Design Criteria

The design criteria of ogee weir are 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. Ogee weir is designed for the stresses in the weir due to external loadings, temperatures, as well as the hydraulic load and other loads applied directly to the structure. Configuration of Ogee weir is shown in **Figure A2.345**.

The design shall satisfy the following requirements of stability:

a) The ogee weirs shall be safe against sliding on any plane or combination of planes within the ogee weir, at the foundation or within the foundation;
b) The ogee weir shall be safe against overturning at any plane within the ogee weir, at the base, or at any plane below the base; and
c) The safe unit stresses in the concrete or masonry of the ogee weir or in the foundation material shall not be exceeded.

B) Codal References

a) IS 456-2000: Plain and Reinforced Concrete - Code of Practice
b) IS 13551 (1992): Structural Design of Spillway Pier and Crest
c) IS 6512: 2019: Criteria for Design of Solid Gravity Ogee weirs

C) Loads

The following forces may be considered as affectingthe design:

a) Dead load,
b) Reservoir and tailwater loads,
c) Uplift pressure,
d) Earthquake forces,
e) Earth and silt pressures,
f) Wind pressure,
g) Wave pressure, and
h) Thermal loads

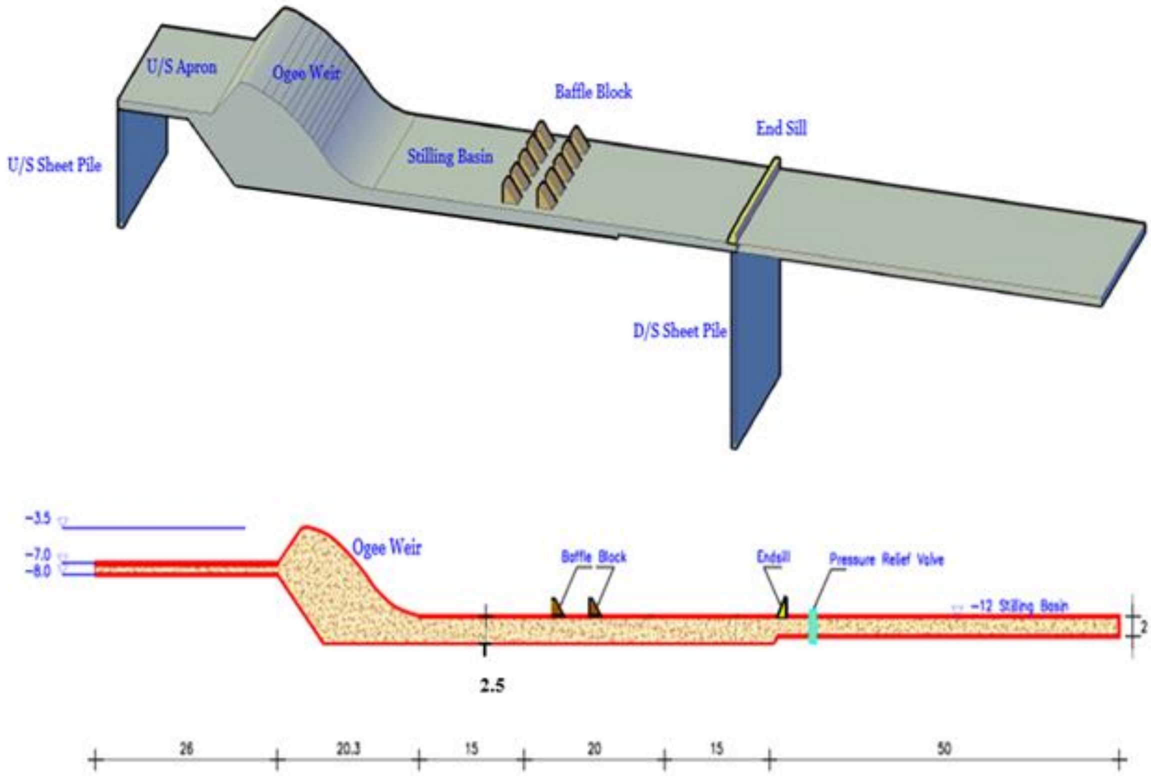


Figure A2.345: Configuration of Ogee Weir

The forces to be resisted by ogee weir fall into two categories as given below:

(i) Forces, such as weight of the ogee weir and water pressure, which can be directly calculated from the unit weights of the materials and properties of fluid pressures; and

(ii) Forces, such as uplift, earthquake loads, silt pressure and, ice pressure, which can only be assumed on the basis of assumption of varying degree of reliability.

For consideration of stability the following assumptions are made:

(i) That the ogee weir is composed of individual transverse vertical elements each of which carries its load to the foundation without transfer of load from or to adjacent elements

(ii) That the vertical stress varies linearly from upstream face to downstream face on any horizontal section.

D) Loads Combinations:

Ogee weir design is designed based on the most adverse load combination A, B, C, D and E given below using the safety factors prescribed.

a) Load Combination A (Construction Condition) — Ogee weir completed but no water in reservoir and no tailwater.

b) Load combination B (Normal operation condition) — Full reservoir elevation, normal dry weather tailwater, normal uplift; ice and silt (if applicable);

c) Load combination C (Flood discharge condition) — Reservoir at maximum flood pool elevation, all gates open, tailwater at flood elevation, normal uplift, and silt (if applicable);
d) Load combination D — Combination A, with earthquake;
e) Load combination E — Combination B, with earthquake but no ice;

E) Design Methodology:

On account of the geometry of spillway crest profile, tensile stresses are developed in the crest because of the loads acting over it. Reinforcement needs to be provided to take care of these tensile stresses. The minimum thickness of structural concrete provided for spillway crest is 1.5 m, measured normally. However, this has to be suitably increased to accommodate the anchorage below the piers. 3D view of STAAD model is shown in **Figure A2.346.**

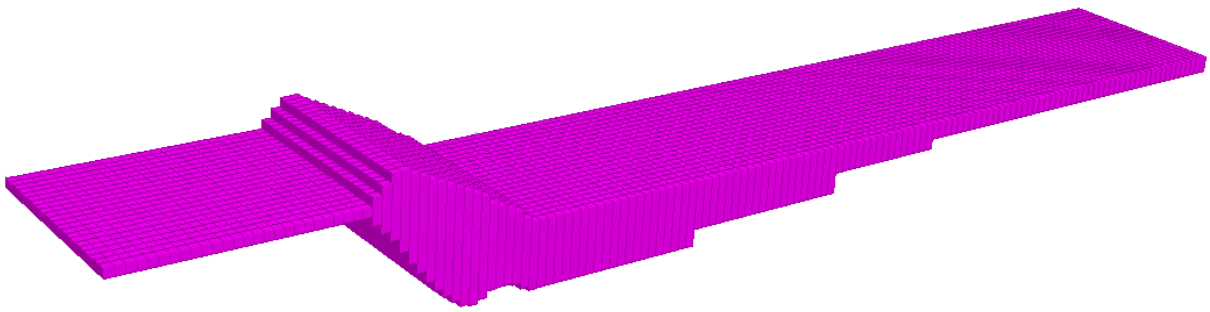


Figure A2.346: 3D STAAD model view of Ogee Weir

F) Load Calculation:

At 1.5 m depth from top of ogee weir:

Dead load of ogee crest

The dead load or the self-weight of this structure is calculated by its unit weight of materials used as per IS875 (part1)-1987- Reaffirmed (2008).

Volume of ogee weir	=	3.23 m^3
Self-weight of ogee weir	=	81 kN
Moment due to self-weight	=	176 kNm

Seismic forces

Horizontal seismic forces	=	24.25 kN
Vertical seismic forces	=	12 kN
Horizontal seismic moment	=	123.45 kN
Vertical seismic moment	=	26.48 kN

Water pressure

Moment due to horizontal water pressure at reservoir side (High flood level) M= 0.16 $\gamma_w H^3$	=	379 kNm
Moment due to horizontal water pressure at reservoir side (Minimum water level)	=	12 kNm
Moment due to horizontal water pressure at sea side (Design water level)	=	1514 kNm
Moment due to vertical water pressure at	=	56.5 kNm

reservoir side (High flood level)		
Moment due to vertical water pressure at reservoir side (Minimum water level)	=	12.5 kNm
Moment due to vertical water pressure at sea side (Design water level)	=	33 kNm

Hydrodynamic pressure

Moment due to hydrodynamic pressure at reservoir side (High flood level)	=	14 kNm
Moment due to hydrodynamic pressure at reservoir side (Minimum water pressure)	=	1.5 kNm
Moment due to hydrodynamic pressure at sea side (Design water level)	=	24 kNm

Weight of gate

Moment due to weight of gate	=	2120.4 kNm

Base pressure

Critical tensile stress is obtained at reservoir (minimum water level) and sea (Design water level).

Total weight	=	1070 kN
Resisting moment	=	2369 kN
Overturning moment	=	1648.5 kN
Eccentricity, e	=	1.6
Width, B	=	4.312
B/6	=	0.72
P/A + M/Z	=	-304 kN/m^2
P/A - M/Z	=	800 kN/m^2

At 2 m depth from top of ogee weir:

Dead load of ogee crest

The dead load or the self-weight of this structure is calculated by its unit weight of materials used as per IS875 (part1)-1987- Reaffirmed (2008).

Volume of ogee weir	=	6 m^3
Self-weight of ogee weir	=	144 kN
Moment due to self-weight	=	419.2 kNm

Seismic forces

Horizontal seismic forces	=	43.1 kN
Vertical seismic forces	=	22 kN
Horizontal seismic moment	=	206.5 kN
Vertical seismic moment	=	63 kN

Water pressure

Moment due to horizontal water pressure at reservoir side (High flood level)	=	469 kNm
Moment due to horizontal water pressure at reservoir side (Minimum water level)	=	22.5 kNm
Moment due to horizontal water pressure at sea side (Design water level)	=	1989 kNm
Moment due to vertical water pressure at reservoir side (High flood level)	=	134 kNm
Moment due to vertical water pressure at reservoir side (Minimum water level)	=	178 kNm
Moment due to vertical water pressure at	=	59 kNm

sea side (Design water level)

Hydrodynamic pressure

Moment due to hydrodynamic pressure at reservoir side (High flood level)	=	24.5 kNm
Moment due to hydrodynamic pressure at reservoir side (Minimum water pressure)	=	3.45 kNm
Moment due to hydrodynamic pressure at sea side (Design water level)	=	42 kNm

Weight of gate

Moment due to weight of gate	=	2827 kNm

Base pressure

Critical tensile stress is obtained at reservoir (minimum water level) and sea (Design water level).

Total weight	=	1167.4 kN
Resisting moment	=	3546 kN
Overturning moment	=	2235 kN
Eccentricity, e	=	2.1
Width, B	=	5.8
B/6	=	0.96
P/A + M/Z	=	-237.2 kN/m^2
P/A - M/Z	=	643 kN/m^2

At 3.5 m depth from top of ogee weir:

Dead load of ogee crest

The dead load or the self-weight of this structure is calculated by its unit weight of materials used as per IS875 (part1)-1987- Reaffirmed (2008).

Volume of ogee weir	=	17.6 m^3
Self-weight of ogee weir	=	440.3 kN
Moment due to self-weight	=	2248 kNm

Seismic forces

Horizontal seismic forces	=	132 kN
Vertical seismic forces	=	66.2 kN
Horizontal seismic moment	=	500.35 kN
Vertical seismic moment	=	337.2 kN

Water pressure

Moment due to horizontal water pressure at reservoir side (High flood level)	=	594 kNm
Moment due to horizontal water pressure at reservoir side (Minimum water level)	=	45 kNm
Moment due to horizontal water pressure at sea side (Design water level)	=	3286 kNm
Moment due to vertical water pressure at reservoir side (High flood level)	=	717.4 kNm
Moment due to vertical water pressure at reservoir side (Minimum water level)	=	581 kNm
Moment due to vertical water pressure at sea side (Design water level)	=	159 kNm

Hydrodynamic pressure

Moment due to hydrodynamic pressure at reservoir side (High flood level)	=	75.2 kNm
Moment due to hydrodynamic pressure at	=	14 kNm

reservoir side (Minimum water pressure)

Moment due to hydrodynamic pressure at sea side (Design water level)	=	130 kNm

Weight of gate

Moment due to weight of gate	=	4949 kNm

Base pressure

Critical tensile stress is obtained at reservoir (minimum water level) and sea (Design water level).

Total weight	=	2570.32 kN
Resisting moment	=	8274 kN
Overturning moment	=	3857.5 kN
Eccentricity, e	=	0.31
Width, B	=	10.1
B/6	=	1.67
P/A + M/Z	=	303 kN/m²
P/A - M/Z	=	208 kN/m²
Maximum tensile stress obtained from worst loading	=	304 kN/m2 < 800 kN/m2, Hence safe
Area of tensile portion	=	182.2 kN
Factored load	=	273.3 kN
Permissible tensile stress in steel	=	220 N/mm2
Area of reinforcement	=	1242.13 mm2

Provide 20 mm dia bar @ 250 mm c/c spacing.

Minimum thickness of ogee crest (as per IS 13551 2019 Cl 4.1) = 1.5

Provided thickness of ogee crest = 1.75 mm

With top 0.3m proposed to be high performance concrete of grade M 50.

G) Design and Analysis:

Load calculation has been carried out considering the above-mentioned loads and load combinations. Design moments have been determined and the tensile stress are computed at every 1.5m as per the mentioned codes. Computed tensile stresses are found to be within the permissible limits as mentioned in table 2 of IS 6512:2019.

H) Results:

As per IS 13551 – 2019, Clause 4.1 minimum thickness of structural concrete provided for spillway crest is 1.5m. From the tensile stress calculations, maximum tensile stress obtained is within the permissible limits. On safety aspect, the thickness of ogee weir crest of 1.5 m is increased by 10 %, therefore provided thickness of structural concrete on weir is 1.75 m. Reinforcement of 20 mm diameter bars @ 250 mm c/c spacing is provided at top 1.75m to counter the calculated tensile stresses. Also, to ensure better hydraulics performance in scouring, top 0.3m is proposed to have a high-performance concrete of M 50 grade.

4) Energy Dissipator

Energy Dissipation is achieved through formation of hydraulic jump under different discharge conditions. For various gate openings with Pond level on the upstream, the discharge through the head regulator and the corresponding water level in the reservoir are worked out. From these values, the cistern levels and lengths are determined in hydraulic design calculation and the governing values are adopted for the profile. Additional energy dissipating devices such as baffle blocks and end sill are provided on the stilling basin (downstream raft) as per the hydraulic design proposal. Configuration of Energy Dissipation system is shown in **Figure A2.347**.

This clause covers the design of the following items

I) Apron Design (incl. upstream apron, downstream apron (Stilling basin) and Secondary Apron);

II) Baffle Block and

III) End sill.

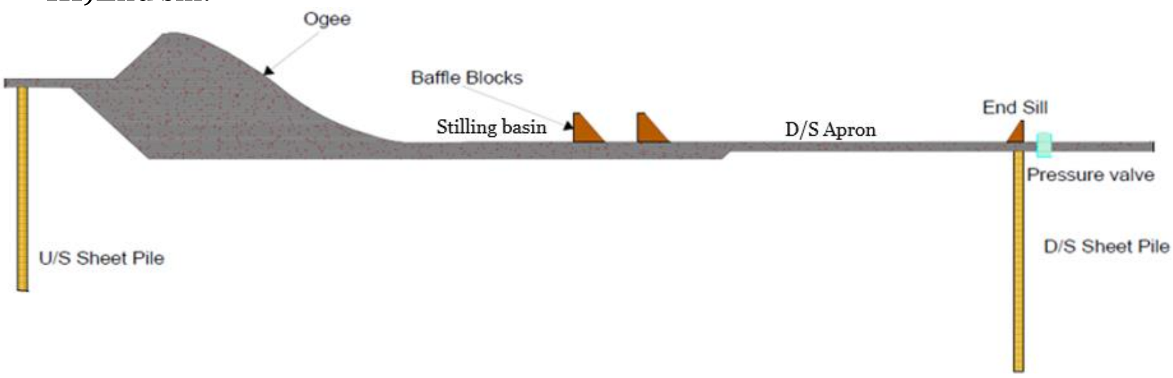


Figure A2.347: – Configuration of Energy Dissipation System

I) Apron Design

A) Design Criteria:

Following are the two types of floors:

i. Gravity type where the uplift pressure is balanced by the self-weight of the floor only considering unit length of the floor;

ii. Reinforced cement concrete raft type where the uplift pressure is balanced by the weight of the floor, piers and other super-imposed dead loads considering a span as single unit.

The design of Reinforced cement concrete raft may generally be done as per the theory of beams on elastic foundation. The design will depend on the value of modulus of subgrade reaction (K), span length, total length of raft etc.

However, for small spans upto 6 m, the floor shall be designed as a continuous beam resting on a homogenous foundation. The abutment, if necessary, may be made independent by providing a joint in the raft with suitable water seals.

B) Codal References:

a) IS 456-2000: Plain and Reinforced Concrete – Code of Practice
b) IS 11527 (1985): Criteria for structural design of energy dissipators for spillways
c) IS 4997 (1968): Criteria for design of hydraulic jump type stilling basins with horizontal and sloping apron
d) IS 1904 (1986): Code of practice for design and construction of foundations in soils

C) Loads:

The raft shall be designed for the moments caused by worst combination of the following forces:

a) Uplift;
b) Soil reaction and;

c) Water Pressure on the raft.

D) Loads Combinations:

Following two extreme conditions may prevail and critical of the two conditions shall be considered for design.

Case(I) – Stilling basin operating during regulator design flood. Water surface over slab at hydraulic jump profile for design discharge, that is flood regulator operating at MWL.

Case (II) – Reservoir at FRL with gates closed when basin is empty.

NOTE – Case I is normally critical and same is considered for basin floor design

E) Design Methodology:

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 shall be designed for the stresses induced due to above forces

F) Design and Analysis:

The slab is divided into independent approximately square panels by contraction joints parallel and perpendicular to channel or basin centre line to avoid serious shrinkage and temperature cracking with the use of nominal reinforcement which does not extend across the joints. The thickness of raft varies on the downstream side (quoted below in table). The support condition is provided as elastic mat with the calculated subgrade modulus.

Table A2.316: Thickness of Raft

Section	Location	Size of raft [L (m)x B(m)]	Thickness of Raft (m)
1	Upstream Apron	26 x 18	1.0
2	Downstream Apron (1st sec)	15 x 22	7.5
3	Downstream Apron (2nd sec)	20 x 22	5.6
4	Downstream Apron (3rd sec)	15 x 22	3.2
5	Secondary Apron	50 x 22	2.0

The panels of slab are reinforced with small amount of steel to prevent harmful cracking resulting from shrinkage and temperature stresses not relieved by contraction joints and on yielding foundations to avoid possible cracking from differential settlement. Usually, a slab on unyielding foundation is reinforced in the top face only to distribute shrinkage cracks and to minimize bending stresses in the slab for the assumed uplift head.

G) Load Calculation

Dead Load

The dead load or the self-weight of this structure is calculated by its unit weight of materials used as per IS875 (part1)-1987- Reaffirmed (2008).

Vertical Water Pressure

Design water level (Sea side)	=	8.1	m
Downstream apron level	=	-12	m

Height of water	=	20.1	m
Unit weight of water	=	10	Kn/m^3
Vertical load due to water pressure	=	201	Kn/m^2

Subgrade Modulus

Safe bearing capacity of the soil under upstream apron	=	420	Kn/m^2
Settlement under upstream apron	=	0.04	m
Subgrade modulus under upstream apron, SBC/ Permissible Settlement	=	10500	$Kn/m^2/m$

Uplift Pressure

Table A2.317: Uplift Pressure

Section on Downstream	Length of Apron (m)	Unbalanced Head (m)	Uplift Pressure (Kn/m^2)
1st Section	15	10.5	75
2nd Section	20	7.5	45
3rd Section	15	4.5	25
Secondary Apron	50	-	-

H) Design Forces and Reinforcement Calculation

The above calculated loads are applied in the STAAD pro (Finite Element Analysis) software. Based on the reactions the reinforcement is calculated. However, the nominal reinforcement for panels on unyielding shall be 20 mm diameter bars at 300 mm centre-to-centre both ways as per IS 11527 (1985): Criteria for structural design of energy dissipators for spillways is found to be sufficient. FEA analysis results of Energy Dissipation system are shown in **Figure A2.348.**

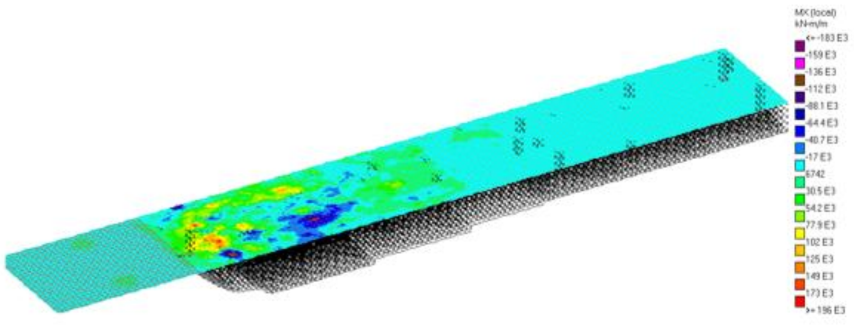


Figure A2.348: Analysis Results of Energy Dissipation System

II) Baffle Block

Location and optimum shape of baffle blocks shall be decided on the basis of IS: 4997-1968. The dimensions of the basin blocks are provided based on the proposal of hydraulic design. The purpose of the block is to dissipate energy and thereby to reduce the length of basin.

A) Design Criteria:

The baffle block is designed for the dynamic force acting on it due to the hydraulic forces on the downstream side.

B) Codal References:

a) IS 456-2000: Plain and Reinforced Concrete – Code of Practice
b) IS 11527 (1985): Criteria for structural design of energy dissipators for spillways
c) IS 4997 (1968): Criteria for design of hydraulic jump type stilling basins with horizontal and sloping apron

C) Loads and Load Combination:

a) Dynamic force against the upstream face of the baffle blocks
b) Negative pressure on the back face of the blocks (which will further increase total load)

D) Design Methodology:

The baffle block is designed manually complying with the Codal provisions mentioned above. Baffle block is designed as a Cantilever structure supported on the raft.

E) Design and Analysis:

Force P acting at $h_b/2 = 2WA\ (D_1 + Hv_1)$

Where,

W	=	Unit weight of water = 10 Kn/m^3
A	=	Area of upstream face of block = 2.6 m^2
$(D1 + hv_1)$	=	Specific energy of the flow entering the basin = 5.9m

From the above calculation,

Force is calculated as 76.7 Kn acting on the baffle at a height of 1.05m from the base of the block. Moment of the block is determined as 80.54 kNm. Area of steel is found to be 20mm diameter at 150mm c/c spacing which should be placed in such a way thatthe reinforcements are tied into the floor slab. Also, all reinforcing steel in a baffle block is placed minimum 150 mm from the exposed surface because of the possible erosive and cavitation action of the high velocity currents.

III) End Sill

The design criteria, loads calculation, design methodology and design are similar to the structural design of baffle block. The dimension details of end sill shall comply with the hydraulic design proposal.

5. Scour Protection

When a firm stratum is not available at reasonable depth below the river bed, toe protection in the form of sheet pile or launching apron may be provided. The sheet pile may be made of RCC or Steel. The sheet piles may be drilled below the river bed up to maximum scour depth. Sheet piles are provided to increase the seepage length under the raft. Launching apron are laid after the rigid raft both upstream and downstream. This clause covers the design of the following items

A) Protection works on upstream and downstream
B) Cut-off wall / Sheet Piles

A) Protection Works

a) Design Criteria:

Concrete apron is designed based on the design concepts for hydraulic structures on permeable foundations i.e., hydraulic structures in rivers. The apron (RCC raft) length is based on the calculations for exit gradient for seepage flow. The design of apron (RCC raft) is covered under the topic “Energy Dissipator”.

b) Codal References:

i) IS 456-2000: Plain and Reinforced Concrete - Code of Practice
ii) IS 6966-1 (1989): Hydraulic design of barrages and weirs - Guidelines

c) Proposed Protection Works:

(i)Cement Concrete Blocks

Just beyond the upstream end of the impervious floor, pervious protection comprising of cement concrete blocks of 1500x1500x900 mm size laid over 600mm thick stone spalls shall be provided. The length of upstream block protection shall be approximately 200m. 75mm gap between blocks filled with bajri/ pea-stones laid over 600mm thick graded inverted filter shall be provided.

On the downstream, hydraulic design proposes an impervious floor of 100m after which the sea bed is dredged from EL(-)12.0 to EL(-)14.5 after a stretch of 10km.

(ii)Loose Stone Launching Apron

Beyond the block protection on the upstream of the impervious floor of flood regulator, launching apron of loose boulders or stones shall be 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 non-availability of stones of specified size, the loose stone apron shall be provided in the form of wire sausages of suitable size. The stone or boulder used shall not be less than 300 mm size and no stone shall weigh less than 40 kg. The quantity of stone provided at upstream of weir shall be adequate to cover the slopes.

B) Cut-off Wall/Sheet Piles

a) Design Criteria:

Considering the merits and demerits of RCC cutoff wall and Steel sheet pile, the later option is preferred considering the ease of execution. Sizes of sheet piles are based on scour phenomenon due to surface flow condition. The sheet pile cutoff is designed as a sheet pile retaining wall anchored at top in the raft and bottom in the substrata. The sheet pile retaining wall will resist the worst combination of forces and moments considering the possible scour on the outer side, earth pressure and surcharge due to floor loads on the inner side and differential hydrostatic pressure computed on the basis of percentage of pressure of seepage flow below the floor. The sheet piles are designed as per CBIP publication No.179.

b) Codal References:
i) IS 456-2000: Plain and Reinforced Concrete - Code of Practice
ii) IS 6966-1 (1989): Hydraulic design of barrages and weirs – Guidelines

c) Design Methodology:
Pressures on the sheet piles are computed considering two possible critical situations:

(i) 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 -5.3 m. Head causing the flow = 8.3 m

(ii) Water level is at +5.0 (MFL); there is flow corresponding to PMF over the spillway and the tail water level is at -1.3 m. Head causing the flow = 6.3 m

d) Design and Analysis:
Pressures considering the above two criteria are computed on the hydraulic design part of report. As per the Hydraulic design proposal the length of Steel sheet pile both on upstream and downstream shall be 11.5m and 12.5m respectively.

Upstream Sheet Pile:
Pressure Calculation:

Surcharge Pressure			
Balanced unit weight	=	14	kN/m3
Surcharge pressure acting on the floor slab, Ka*ϒ*h	=	5.39	kN/m2
Active Earth Pressure			
Submerged unit weight of soil	=	9	kN/m3
Height of wall/earth pressure acting on wall	=	11.50	m
Pressure on wall, Pa=Ka*ϒ*h	=	40.16	kN/m2
Net pressure acting on wall	=	45.55	kN/m2
Rate of change of pressure, K	=	20.69	kN/m2
a = Net pressure/Rate of change of pressure	=	2.20	m

Section Modulus:

Permissible tensile stress in steel , 0.6*fy N/mm2 = 150
Section modulus required Zreq cm3 = 5285.95

Downstream Sheet Pile:
Pressure Calculation:

1) Surcharge Pressure

Balanced unit weight	=	14	kN/m3
Surcharge pressure acting on the floor slab, Ka*ϒ*h	=	10.78	kN/m2

2) Active Earth Pressure

Submerged unit weight of soil	=	9	kN/m3
Height of wall/earth pressure acting on wall	=	12.50	m
Pressure on wall, Pa=Ka*ϒ*h	=	43.65	kN/m2
Net pressure acting on wall	=	54.44	kN/m2
Rate of change of pressure, K	=	20.69	kN/m2
a = Net pressure/Rate of change of pressure	=	2.63	m

Section Modulus:

Permissible tensile stress in steel , 0.6*fy	=	150	N/mm2
Section modulus required Zreq	=	5285.95	cm3

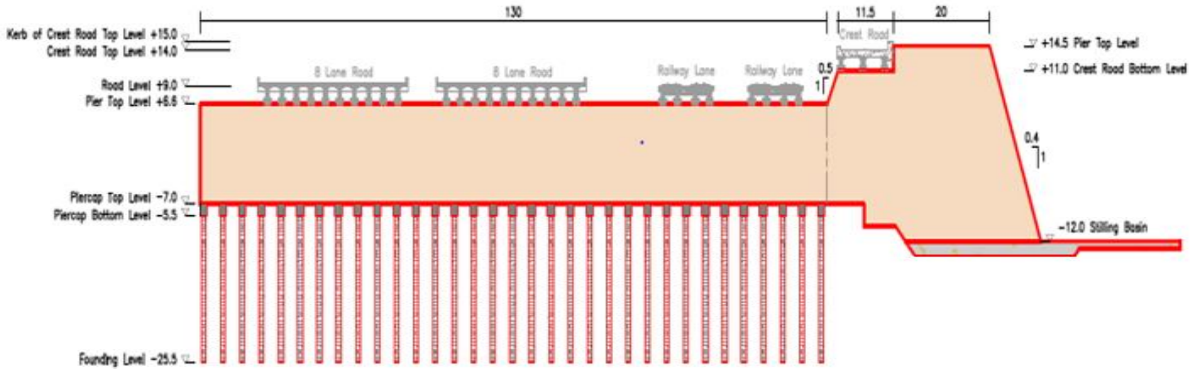
e) Results:

Provide "AZ 46-700N" profile steel sheet pile from Arcelor Mittal Sheet Piling catalogue both upstream and downstream, whose section modulus value is 5350cm3/m and thickness would be 20mm.

6. Pier

A) Design Criteria:

Kalpasar Dyke will have a flood regulator opening for about 2.2 km stretch. There are 100 bays and each bay is separated by piers. Wall Pier of 4m thick is proposed at about 100 nos. In the downstream side, wall piers support ogee weir, breast wall, vertical lift gates and its associated hydro-mechanical components and resist the uplift pressure. Pier acts as a support system for the transportation corridor on the upstream side. Pier on the upstream and downstream side is connected through construction joints. Wall pier on the downstream side is designed as a plate element in staad pro and is analysed for the loads and load combinations mentioned below. Crest Road used for maintenance of gates is modelled separately and reactions are applied on the wall pier. Wall pier is designed as supporting over raft foundation at bottom. Soil conditions are modelled as elastic mat with sub-grade modulus. Configuration of Wall pier is shown in **Figure A2.349**.



FigureA2.349: Configuration of Pier

B) Codal References:

a) IS 456-2000: Plain and Reinforced Concrete - Code of Practice
b) IS 13551- 2019 – Structural Design of Spillway Pier and Crest- Criteria.
c) IRC 6 - Standard Specifications and Code of Practice for Road Bridges.

C) Loads:

a) Self-weight of pier (DL);
b) Crest Road reaction on the pier (ILL);
c) Weight of hoisting equipment and gate on the pier (GL);
d) Breast Wall load (BL);
e) Transverse water pressure on the pier (H_yS);
f) Hydro-dynamic Load (H_yD);
g) Current Load (CL);
h) Wave Load ($W_{av}L$);
i) Wind load (WL);
j) Earthquake Load (EQ);

D) Loads Combinations:

Table A2.318: Load combinations considered for Design

Load Cases	**DL**	**ILL**	**GL**	**BL**	**WL**	**HyS**	**HyD**	**CL**	**WavL**	**EQ**
Dry Case: Non Seismic Normal Loading	1.5	1.5	1.5	1.5	1.5	-	-	-	1.2	-
Dry Case: Seismic Normal Loading	1.5	-	1.2	1.2	-	-	1.5	-	1.2	1.5
Dry Case: Extreme	1.2	-	1.2	1.2	-	-	1.2	-	1	1.2
Dry Case: Reversal	0.9	0.9	0.9	0.9	1.5	-	-	-	1	1.5
Wet Case: Non Seismic Normal Loading	1.5	1.5	1.5	1.5	1.5	1.2	-	1.2	1.2	-
Wet Case: Seismic Normal Loading	1.5	-	1.2	1.2	-	-	1.5	1.2	1.2	1.5
Wet Case: Extreme	1.2	-	1.2	1.2	-	1	1.2	1	1	1.2
Wet Case: Reversal	0.9	0.9	0.9	0.9	1.5	1	-	1	1	1.5

E) Design Methodology:

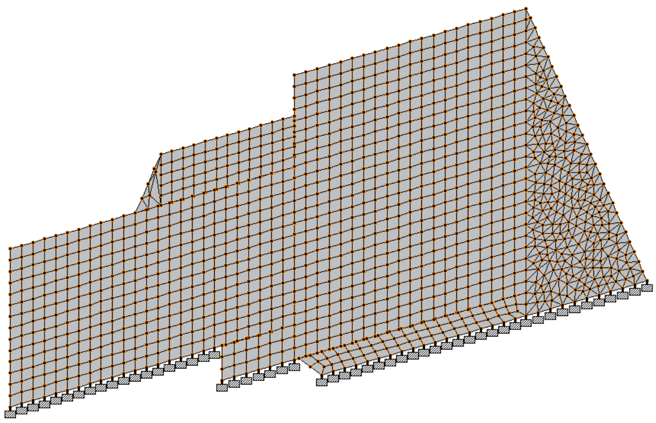
3D view of STAAD model is shown in **Figure A2.350**. Pier and associated foundation system is analysed for following cases:

a) Dry Condition:

Basically, the hydro-mechanical gates are considered to be closed in this case and there is no water flow at this stage of analysis. First pier structure is analysed for static load with Dead load, deck load, wind load, and loads due to breast wall and vertical lift cases. Analysis with same loads is again carried out for combination of seismic loads

b) Flooded Condition:

Hydro-mechanical gates are considered to be open and water flow will be there in bays. In additional to above mentioned static loads, water currents, hydrostatic loads and buoyant uplift forces are considered for this analysis. Same load condition is checked for combination of seismic loads along with hydrodynamic loadings



FigureA2.350: 3D STAAD model view of Wall Pier near Ogee

F) Load Calculation:

Dead Load:

The dead load or the self-weight of this structure is calculated by its unit weight of materials used as per IS875 (part1)-1987- Reaffirmed (2008).

Crest road (ILL)

Staad analysis of crest road is done, reactions are taken from the analysis.

Seismic case:

F_y = 2270 kN

F_y = 2266 kN

F_y = 2270 kN

Non-Seismic case:

F_y = 4229 kN

F_y = 3439 kN

F_y = 3257 kN

Gate and gantry weight (GL)

Gate Weight in one Pier	=	590.5	kN
Gate Weight in one Pier per m2	=	6.71	kN/m2
Gantry Crane Weight	=	80	t

	=	800	kN
Gantry Crane Weight per m2	=	9.09	kN/m2
Breast Wall weight (BL)			
Breast wall Top level	=	14.5	m
Breast wall bottom level	=	6.5	m
Thickness	=	0.55	m
Span	=	18	m
Weight	=	1980	kN
Weight on Single Pier	=	990	kN
Wind (WL)			
Wind speed	=	50	m/s
Risk coefficient factor	=	1.08	
Factor	=	1.09	
Topography factor	=	1	
Cyclone importance factor	=	1.15	
Design wind speed	=	$V_b * k_1 * k_2 * k_3 * k_4$	
	=	67.689	m/s
Design wind pressure	=	$0.6 * Vz^2$	
	=	2.749080433	kN/m2
Gust Factor	=	2	
Drag coefficient	=	1.1	
Ft	=	6.047976952	kN/m2
Transverse and longitudinal water pressure (Reservoir - high flood level - + 5 m) (HyS)			
Water level (Reservoir side)	=	5	m
Reservoir bed level	=	-7	m
Height of water	=	12	m
Water pressure	=	123	kN/m^2
		123	kN/m^2
Transverse and longitudinal water pressure (Sea - Design water level - + 8.1 m)			
Water level (Sea side)	=	8.1	m
Sea bed level	=	-12	m
Height of water	=	20.1	m
Water pressure	=	206	kN/m^2
		206	kN/m^2
Transverse and longitudinal water pressure (Reservoir - Tail water level - -4 m)			
Water level (Reservoir side)	=	-4	m
Reservoir bed level	=	-7	m
Height of water	=	3	m
Water pressure	=	30.75	kN/m^2
	=	30.75	kN/m^2
Transverse and longitudinal water pressure (Sea - Low tide level - -5.3 m)			
Water level (Reservior side)	=	-5.3	m
Sea bed level	=	-12	m
Height of water	=	6.7	m
Water pressure	=	68.68	kN/m^2
	=	68.68	kN/m^2

Water current (CL)

Current Intensity, P = $52KV^2$

Where,

P = intensity of pressure due to water current, in kg/m2

V = the velocity of the current at the point where the pressure intensity is being calculated, in meter per second, and

K = a constant dependent on shapes of piers, for circular piers or piers with Semi-circular ends 0.66.

P = 875 kg/m^2 = 8.75 kN/m^2

Earthquake loads:

Earthquake loading as applied in the structure by using seismic definition in Staad Pro Software relying to the standard IS 1893 Part 1 -2016 in seismic coefficient method. Currently, the India Seismic Code IS 1893 (Part 1) is under revision. As per the new standard the location of the dyke falls in Zone IV against Zone III in existing standard. The safety assessment level taken for dyke (0.36g) is considered for structural design for flood regulator (corresponding to a return period of 975 years). The acceleration corresponding to the extreme event is taken two levels higher as 0.54g (corresponding to a return period of 4995 years). Also, the return period for the serviceability check is taken one level lower as 475 years; the associated acceleration value is 0.30g.

Wave pressure (Wav L)

As per IS 4651, Annexure B-1

Depth of water	=	18.5	m
Height of free wave	=	2	m
weight of water	=	10	kN
Time Period	=	12	s
Deep water wave length Lo	=	224.64	m
d/Lo	=	0.082353989	
d/L	=	0.125	As per wave table
L	=	148	
cosh2pid/L	=	0.707106781	
Clapotis wave pressure	=	28.28427125	kN/m2
	=	30	kN/m2

Hydrodynamic (HyD)

P = $C_s \alpha_h wh$

Where,

P = hydrodynamic pressure in kg/ma at depth y,

C_s = coefficient which varies with shape and depth

α_h = design horizontal seismic coefficient

w = unit weight of water in kg/m^3, and

h = depth of reservoir in m

For maximum water level

Downstream

Cm	=	0.58	(From figure 10 of IS 1893-1984)
Cs	=	0.58	Maximum condition
Unit weight of sea water	=	10.25	kN/m²
Height of water	=	20.1	m
Zone factor, Z		0.3	Zone 3 revised IS 1893
Importance factor, I	=	1.5	
Response reduction Factor , R	=	3	omrf
Sa/G	=	2.5	
Horizontal seismic coeff Ah	=	0.1875	
	=		
P	=	22.40521875	kN/m²
Upstream			
Cm	=	0.7	(From graph)
Cs	=	0.7	Maximum condition
Unit weight of sea water	=	10.25	kN/m²
Height of water	=	12	m
Zone factor, Z	=	0.3	Zone 3 revised IS 1893
Importance factor, I	=	1.5	
Response reduction Factor , R	=	3	omrf
Sa/G	=	2.5	
Horizontal seismic coeff Ah	=	0.1875	
P	=	16.14375	kN/m²
For minimum water level			
Downstream			
Cm	=	0.58	(From figure 10 of IS 1893-1984)
Cs	=	0.58	Maximum condition
Unit weight of water	=	10.25	kN/m²
Height of water	=	6.7	m
Zone factor, Z	=	0.3	Zone 3 revised IS 1893
Importance factor, I	=	1.5	
Response reduction Factor , R	=	3	omrf
Sa/G	=	2.5	
Horizontal seismic coeff Ah	=	0.1875	
	=		
P	=	7.468	kN/m²
Upstream			
Cm	=	0.7	(From graph)
Cs	=	0.7	Maximum condition
Unit weight of water	=	10.25	kN/m²
Height of water	=	3	m
Zone factor, Z	=	0.3	Zone 3 revised IS 1893
Importance factor, I	=	1.5	
Response reduction Factor , R	=	3	omrf

Sa/G	=	2.5
Horizontal seismic coeff Ah	=	0.1875
	=	
P	=	4.036 kN/m²

Buoyant uplift (BL)

Downstream

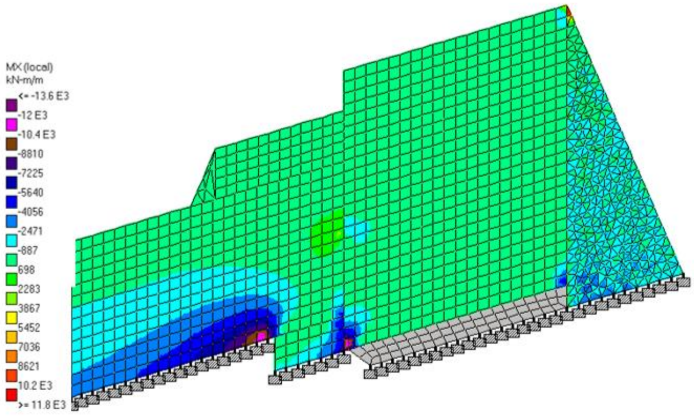
Buoyant uplift	=	Water level up to HTL * unit weight of sea water
	=	206.025 kN/m²

Upstream

Buoyant uplift	=	Water level up to HFL * unit weight of sea water
	=	123 kN/m²

G) Design Forces and Reinforcement Calculation:

Wall pier is modelled and analysed as a plate element in STAAD pro for the above-mentioned loads and load combinations. Design moments are extracted from the analysis and reinforcements are calculated for the same. 4m thick pier with 0.4% of reinforcement in horizontal face and 0.2% of reinforcement in vertical face are found to be satisfied. As the pier and ogee weir will be constructed monolithically, some proportion of tensile stress will get transferred from pier to ogee weir and vice versa. Therefore, appropriate anchorage reinforcements are provided which is about 0.2% as per SP-55:1993. Analysis results are shown in **Figure A2.351**.



FigureA2.351: Analysis Results of Wall Pier near Ogee

7. Stability Analysis

A) Design Criteria:

Flood regulator consists of components such as cutoff/ sheet pile, Raft, piers, abutments, flank wall, u/s and d/s protection work, road bridge, hydro mechanical accessories, guide bunds. Stability analysis is an important part of planning and design of any diversion structure (Barrage or Weir). Before the individual components of diversion structure are designed structurally for various design conditions, its stability as a whole is checked. The design conditions and forces are dependent upon the configuration of the diversion structure i.e., whether it is:

a. Independent pier with gravity floor (resisting forces mainly through gravity)
b. Raft (resisting forces through slab /beam action)

For independent pier with gravity floor type structure, each pier and abutment are structurally separate from the floor through PVC seals. The loads from Pier do not get transferred to floor. Therefore, stability of every Pier / Abutment is checked independently. Stability analysis for such structures is very important. Sizes and thickness of various components are determined to resist the loading.

For Raft type structure, the piers and / or abutments are monolithic with floor and floor also participates in resisting the forces by slab / beam action. Raft structures are more stable against sliding or overturning. However, floatation condition is checked for raft structures. Sometimes, due to design considerations, the raft and gravity both type of components are provided. Under such case, the stability of all individual component is checked.

B) Codal References:
a) IS 456-2000: Plain and Reinforced Concrete - Code of Practice

C) Components Considered in Stability Check:

a) RCC Raft
b) Pier
c) Ogee weir
d) Weight of water at FRL [EL (+3.0)] on u/s
e) Weight of water in stilling basin at TWL [EL (-5.3)]
f) Weight of water at HFL [EL (+5.0)] on u/s
g) Uplift at FRL [EL (+3.0)] with TWL [EL (-) 5.3]
h) Uplift at HFL [EL (+5.0)]
i) Water pressure under the raft at FRL [EL (+3.0)] - No flow condition
j) Water pressure under the raft at HFL [EL (+5.0)]
k) Transportation Corridor
l) Gate House

D) Design Methodology:
a) Non-Earthquake Condition
  i) No Flow Condition-With No Water in Stilling Basin
  ii) No Flow Condition-With Water in Stilling Basin @ EL (-)5.3m
b) Earthquake Condition
  i) No Flow Condition-With No Water in Stilling Basin
  ii) No Flow Condition-With Water in Stilling Basin @ EL (-)5.3m
c) HFL Condition

E) Stability Check:
Stability check has been carried out considering the all the components acting as one single component. The component considered for stability and the methodology adopted for checking are quoted above. The factor of safety against sliding and overturning under different conditions are mentioned in the table below.

Table A2.319: Factor of Safety

Condition		Factor Of Safety	
		Sliding	Overturning
Non-Earthquake Condition	No Flow Condition-With No Water in Stilling Basin	13.84	6.80
	No Flow Condition-With Water in Stilling Basin @ EL (-)5.3m	14.48	7.41
Earthquake Condition	No Flow Condition-With No Water in Stilling Basin	3.02	6.86
	No Flow Condition-With Water in Stilling Basin @ EL (-)5.3m	3.16	7.47
HFL Condition		2.04	8.97

As per IS codal provision IS-456-2000, the factor of safety against sliding and overturning are limited to 1.4 and 1.2 respectively. Computed FOS in stability analysis falls within the permissible limits.

2.3.9 Construction Sequence

The Flood regulator is located in the intertidal region on Dahej side with the ground level varying between +3 to +4m MSL. The length of the flood regulator is approximately 2.2km. The predominant soil in the region is silty sand followed by hard to very stiff clay.

There will be an approach and tail end channel on the upstream and downstream of the regulator, respectively. The channels will extend in such a way that it ends at -7m MSL on the upstream side and -8m MSL on the downstream side. The flood regulator will be constructed independent of dyke construction. The cross section of spillway is as per the given **Figure A2.352** The sequence of construction of the spillway is explained below.

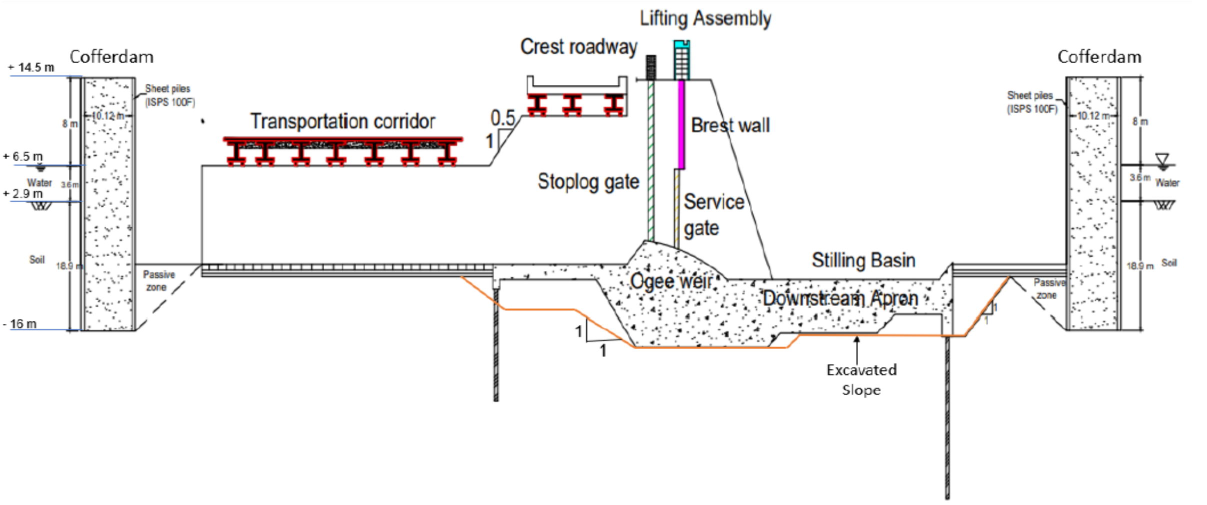


Figure A2.352: Cross section of Spillway and Cofferdam

Stage 1 - Construction of Cofferdam, Dewatering and Dredging:

The spillway has to be constructed only after the construction of cofferdams on upstream and downstream sides. Cofferdams restrict the water from entering into the construction area. The water present within the coffer dams can be dewatered before construction and the bed can be excavated up to the level of the stilling basin. Based on the depth of water and soil to be retained, type of coffer dam to be constructed is selected. Since the depth of soil and water to be retained in this case is more than 15 m, cellular cofferdam is suitable for this case.

Sheet Piles for caissons are founded in the soil up to -16m depth. Once the cofferdams are positioned suitable drain points, and dredging equipment, pumping systems are installed at the required locations inside the Cellular cofferdam. Water inside the cofferdam is pumped out simultaneously from all the corners. After draining, dredging equipment will dredge up to -7m in both upstream and downstream side. Subsequently dredging of upstream and downstream channels upto required depth will be done. In the downstream side, still basin is located at 12m depth and the base slab is about 12m thick. The remaining required depth in downstream side will excavated using suitable mechanism.

Stage 2 – Construction of Bottom Raft on downstream side:

After excavation, Compaction of soil layers with either chemical stabilisation or PVD is done to make the foundation strong enough. Following that the floor is properly levelled and the soft and weathered materials are removed. Shuttering works for the concreting of raft is done. It should be rigidly constructed and efficiently propped and braced both horizontally and vertically, to retain its shape. The joints in the formwork should be tight against leakage of cement grout. Mix design concrete must comply with Indian Standards for Marine exposure conditions. Each batch RCC mix should be checked for its characteristic compressive strength with 3 samples. Cube compressive strength should be performed daily and records should be properly done. First lift of RCC is done and the corresponding immediate settlement is measured. Concreting of Raft slab up to -12m is done subsequently. After length of stilling basin, the depth of stilling basin is slowly raised to -8m to match with contours of downstream channel.

Stage 3 – Construction of pier foundation on Upstream side:

Pier will be founded with series of piles and raft system. Proper equipment for boring pile in marine soil conditions must be ensured. Cast in-situ or precast pile boring will be done. Piles in each row will be connected through pile cap above which the pier will rest and distributes the load. Piles are founded to about 20m length. After concreting pile cap will form the top level of -7m in upstream side. Suitable concrete pedestals are installed above the pile cap to ensure proper load distribution between pier and pile foundation.

Stage 4 – Construction of Weir and Energy Dissipation System:

Weir/chutes shall be provided to act as a control structure on the flood regulators. Ogee shaped Weir is made as a Mass concrete structure, with minimal reinforcement. So, the sulphate and alumina content in cement should be checked for mass concreting conditions. Weir is concreted from -7m level to -3.5m level, the crest. Energy dissipating system shall be provided in the sea side to convert super-critical flow into sub-critical flow to prevent erosion in the sea side of the flood regulator. They are made of Baffle block systems with End sill, made of concrete. For pervious protection works, cement concrete blocks and loose stone apron/boulders shall be laid after the impervious raft foundation in the sea side of the flood regulator. These blocks are kept at -8m level which will be elevated than actual raft thickness.

Stage 5 – Construction of Pier and Transportation corridor:

Piers are wall mounted type with 4m thickness. Piers are constructed as RCC wall from -7m to +6.5m, by stage wise lift construction. Construction joints should be provided at various stages of concreting. Expansion joints with suitable material should be provided every 30m. Structural stability of Shuttering units should be checked and additional braces and stiffeners are given to increase the stability. After Pier, Suitable precast I girders are laid with connecting beams. Transportation corridor will be constructed as rigid pavement supported over those I girders. Crest roadway also subsequently done up to an elevation of +14.5m

Stage 6 – Construction of Retention structures and channel:

Suitable retention structures like abutment/guide bund/flank wall shall be designed to retain the earth fill along the sides of the flood regulator. Abutment will act as an interface between earthen embankment and concrete spillway. Proper temporary retention structures like struts with braces should installed to prevent sliding of soil, during the construction phase. Once the concreting of abutment is done, flank wall construction will start with subsequent dredging of channel. Dredging of channels upstream and downstream of the flood regulator is done using Bull Dozer, hydraulic excavators, or rainbow dredgers, or pumping. Slope protection with stone pitching will be done, when dredging is carried out in stages to avoid collapse of the channel sides. Then the placement of concrete blocks and filter layer on the base of the channels will be carried out wherever required as per design

Stage 7 – Installation of Hydro mechanical Components

Hydro-mechanical gates will rest on the control structure, to control/regulate the flow of water on the reservoir side. Initially breast wall construction will start from +6.5m elevation to +14.5m level. Breast walls will be supported on either side by wall Piers. Followed this, the wheel track made of steel girders will be installed. Service gate is then installed using proper gantry cranes. Stop logs are racked and will be used at time of need.

2.3.10 Instrumentation

2.3.10.1 Sensors for Structural Parameters

a) Movements

The abutment blocks, divide walls, flank walls are tall and isolated structures that may undergo tilt particularly in seismic regions or where deep scour and differential pressures are expected on these structures. Instruments like plumb lines or inclinometers or a set of them can be installed for monitoring the tilts. The deflection of the flood regulator components is the only single parameter affected by all the loads on the flood regulator foundation system and, therefore, is an important aspect that would give a significant measure of deflections.

b) Internal Joint Movement

Flood Regulators are generally built-in blocks separated by transverse joints. It is essential to monitor any relative movement between the blocks. The movement is likely due to differential foundation behaviour. The devices for monitoring joint movement are (a) unbonded resistance wire type, and (b) vibrating wire type. Joint movement meters should be provided in between all blocks. The rigidity/flexibility of the block should be the deciding factor for the spacing along the joint.

c) Surface Joint Movement

Surface joint movement are monitored either on the surface or at locations accessible from galleries. The measurements are made by calibrated tapes by fixing two reference points one each side of the joint and by accurately measuring the distance between the two points at desired intervals. Measurement of joint movement at the surfaces which are accessible from galleries and/or exterior surface of the dam may be monitored by using detachable gauges. Portable gauges with dial indicators and crack meters are most common mechanical gauges for measurement of surface movements. 1D, 2D and 3D measurements are possible. These devices should be located across the known joints. The surface joint measurements are also taken where surface cracks are noticed.

d) Tilt

Tilt is measurement of rotation in vertical plane. It is normally measured with the help of tilt meter system consisting of tilt meter sensor, tilt plates and indicator. Tilt plates are bonded to the surface of mass of structure under observation. The sensor is oriented on three pegs of tilt plate and senses change in tilt of tilt plate. The portable indicator gives the degree of rotation. Tilt measurements are available as vibrating wire inclinometers. Vibrating wire type inclinometers are either surface mounted or embedded in the body of dam. A cylindrical core houses a special pendulum surrounded by damping oil. A vibrating wire is stretched between the pendulum and the core. The instrument works on the principle that any change in the position of pendulum will change the tension in the vibrating wire and its frequency of vibration will change. The change in the frequency of vibration of the wire is calibrated with the tilting of the instrument. The sensors/instruments for the structural parameters were given in **Table A2.320**.

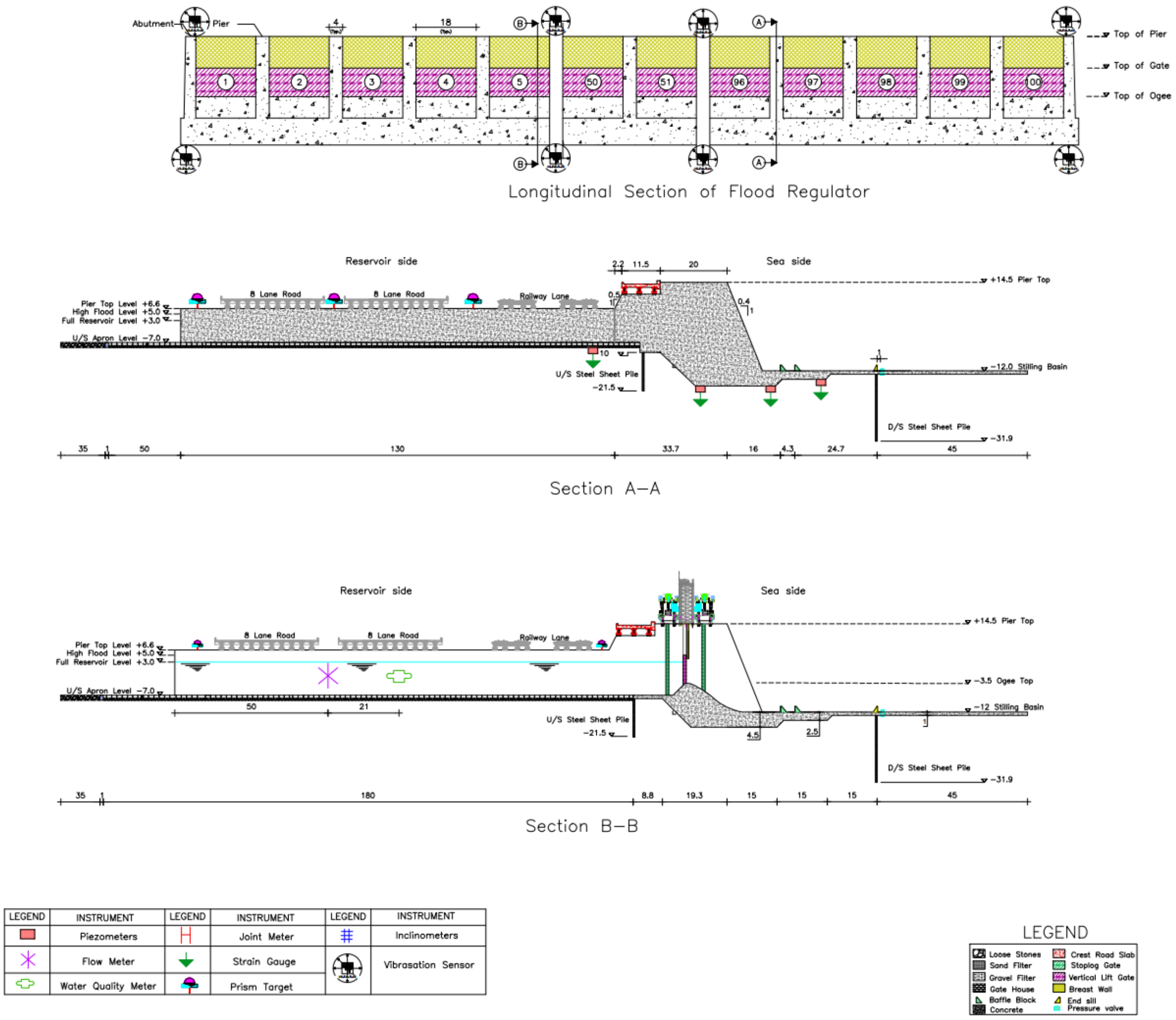


Figure A2.353: represents the proposed locations of piezometer, joint meter, strain gauge, prism target to be installed

Table A2.320: List of structural parameters and its sensors for measurement and monitoring

Structural Parameters	Sensor / Instrument	Processing Data Frequency
Pore Pressure	Piezometers	Daily
Internal Joint Movement	Bi-axial crack meter	Daily
Surface Joint Movement	Tri-axial crack meter	Daily
Relative Movement between Parts of Dam	Prizm Target	Daily
Tilt	Inclinometer	Daily

2.3.10.2 Sensors for Seismic Parameters

a) Vibration

As per recommendations of IS 4967, a seismological observatory for the entire project is to be established at the project site for evaluation of seismic parameters for large dams. The location of observatory should be so selected that the firm rock or suitable ground is available for founding the instrument pillars. The observatory is to be located at site which is unaffected by vibrations caused by powerhouse operations, etc.

Strong motion accelerographs and structural response recorders are to be installed at the base of the dam (in a recess provided in the foundation gallery and at the top of the dam). The location may be suitably selected to avoid the background seismic noise created due to the vibration originating from the appurtenant works of the dam. The instruments located in the foundation gallery are meant for observing the input ground motion in the event of major earthquake. The instruments located at the top of the dam are expected to provide information about response of the structure to the earthquake.

b) Ambient Vibration

Ambient vibration tests use the response of the dam under ambient sources of excitation, such as wind, vehicular motion, machinery operating in the vicinity, and flow of water into the reservoir and over the spillways. The level of response due to ambient vibration is small, in general. But it can be captured using sensors having high sensitivity. Also, under normal conditions of the dam and under ambient vibrations, the response of the dam is in the linear elastic range. As the input excitation due to ambient vibration is random in nature, the structure is always in its transient state of response, and hence, vibrates at all its natural frequencies. Further, considering that the ambient excitation has fairly uniform energy distribution over all the frequencies, the fundamental mode will dominate the response, as it requires the least amount of energy for excitation.

c) Forced Vibration

Forced vibration test is based on the concept of resonance. When a structure is excited by an external vibratory force (harmonic, random, or impact) at the frequency near its natural frequency, the response of the structure is amplified significantly. The dynamic harmonic force is applied using an eccentric mass shaker on the crest of the dam and the sensors placed on the down the downstream face. The dam is excited at its natural frequencies; the frequency of harmonic excitation can be changed gradually in small steps within the range of the natural frequencies of interest to obtain the frequency sweep response of the dam. At each resonant state corresponding to a natural frequency, the response will indicate a local peak. These frequency response curves can be used to obtain the natural frequency and damping. Also, at each natural frequency, the shape of the deformation of the dam is captured by the relative values of the lateral response of the dam recorded by the sensors on the downstream face of the dam. In this manner, the dynamic characteristics of the dam are obtained.

d) Traffic vibration

The traffic induced vibrations on dam strength and concrete compressive strength effects to be studied for full-depth to measure the structural performance. The traffic induced vibrations are not detrimental to the quality of repair concrete when low slump concrete is used and the reinforcing bars are securely fastened to the structure before the concrete placement.

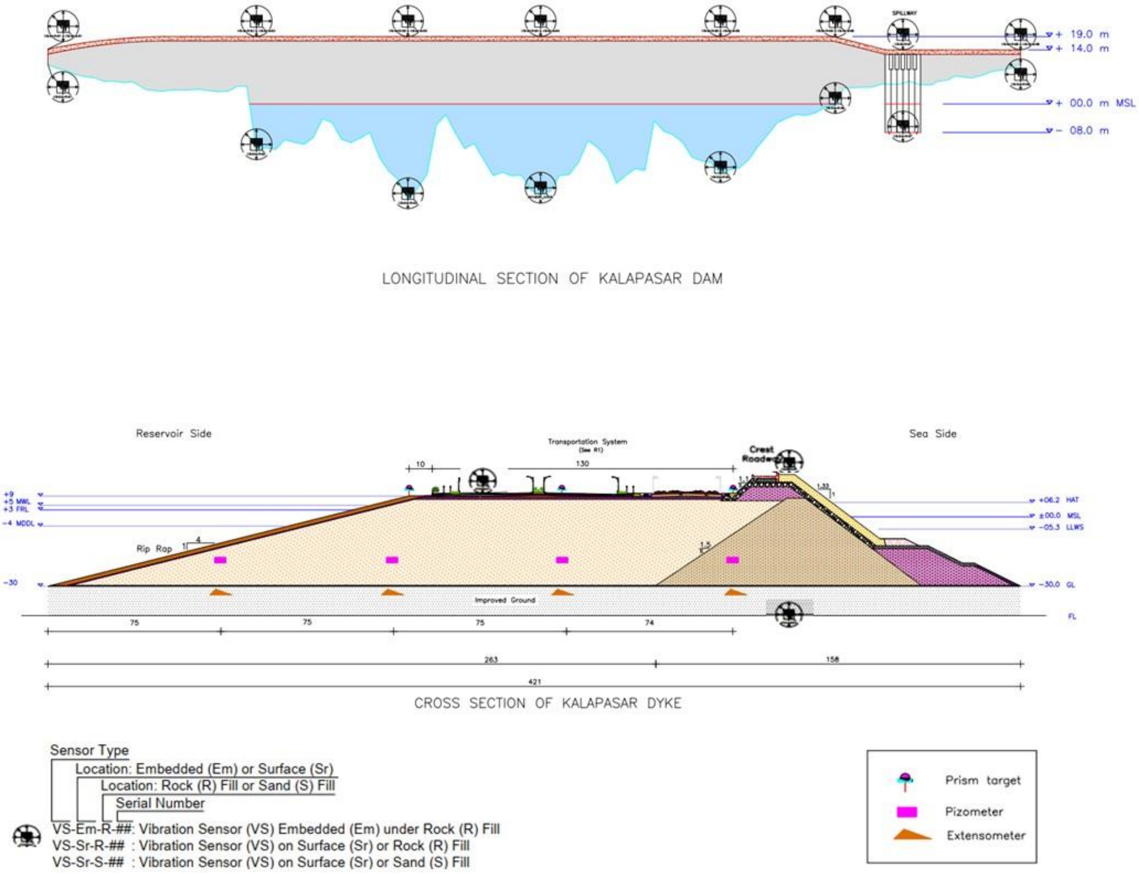


Figure A2.354: Proposed Locations of Seismic Vibration Sensor to be Installed

e) Seismic Vibration

Seismic instruments have to be installed prior to construction to record tremors including micro-tremors in and around the project area and in the embankments to measure the effects of seismic events that occur subsequent to the construction. Data thus collected would also help to take a view on the question of induced seismicity in the area. The seismic instruments provide seismic data such as acceleration, velocity and displacement.

As per recommendations of IS 4967, a seismological observatory for the entire project is to be established at the project site for evaluation of seismic parameters for large dams. The location of observatory should be so selected that the firm rock or suitable ground is available for founding the instrument pillars. The observatory is to be located at site which is unaffected by vibrations caused by powerhouse operations, etc. It is proposed to place the seismological observatory will be at the top of the dam as shown in **Figure A2.354**.

i) Acceleration

Strong motion accelerographs and structural response recorders are to be installed at the base of the dam (in a recess provided in the foundation gallery and at the top of the dam. The location may be suitably selected to avoid the background seismic noise created due to the vibration originating from the appurtenant works of the dam. The instruments located in the foundation gallery are meant for observing the input ground motion in the event of major earthquake. The instruments located at the top of the dam are expected to provide information about response of the structure to the earthquake.

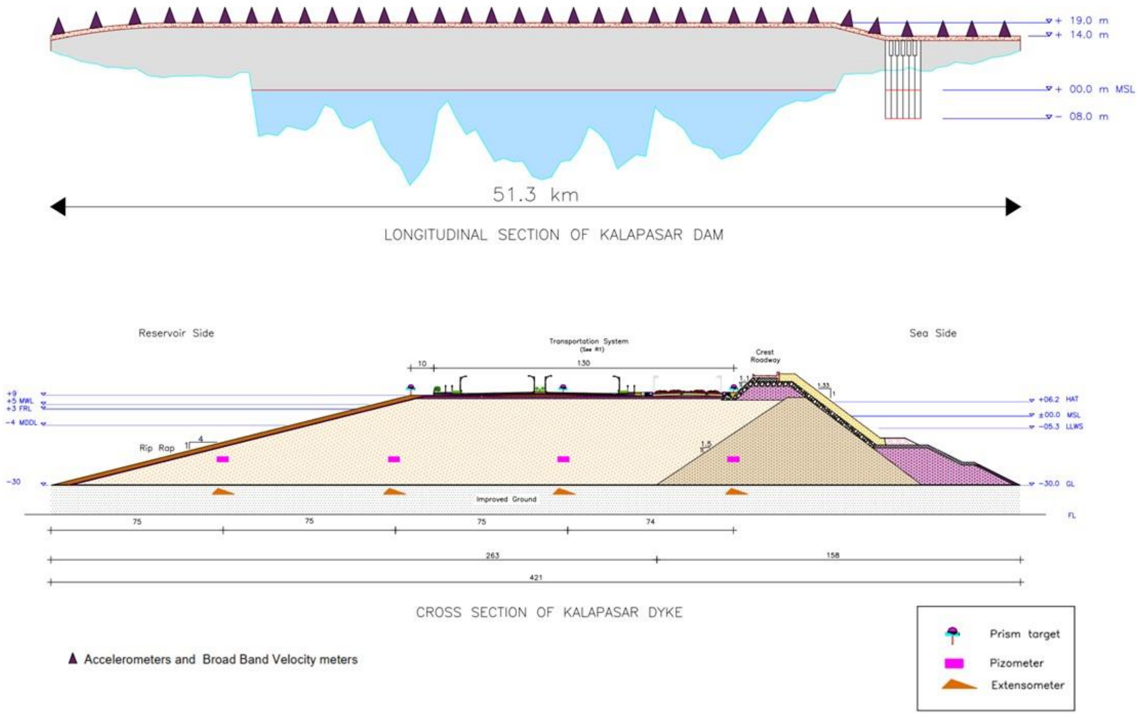


Figure A2.355: Proposed Locations of Accelerometer and Velocity-meter

ii) Velocity

Seismic velocity is the speed with which an elastic wave propagates through a medium. For non-dispersive body waves, the seismic velocity is equal to both the phase and group velocities; for dispersive surface waves, the seismic velocity is usually taken to be the phase velocity. Seismic velocity is assumed usually to increase with increasing depth and when measured in a vertical direction it may be 10–15% lower than when measured parallel to strata.

iii) Displacement

Seismic displacements have a horizontal and a vertical component. The horizontal component is often clearly visible along fault lines activated by the earthquake, though more or less regular horizontal deformation may extend along the surface of nearby crustal blocks. In coastal areas, the vertical component is easily measurable and most important from a geodetic point of view, because it changes the relation to sea level. The sensors/instruments for the seismic parameters were given in **Table A2.321**.

Table A2.321: List of Seismic Parameters and its Sensors for Measurement and Monitoring

Seismic Parameters	Sensors & Instruments	Data Sampling Frequency
Ambient vibration	Triaxial Accelerometer	Daily
Acceleration	Accelerometer with Recorder	Daily
Displacement	Strong Motion Accelerograph with Recorder	Daily
Velocity	Broad Band Velocity Meter	Daily

2.3.10.3 Sensors for Hydrological Parameters

a) Water Level

The water level at the upstream side of the spillway is at +3m for full reservoir level while during flooding, the maximum Flood level is at +5m, beyond which the water is released. The measurement of water levels on the upstream and downstream of dam is useful for calculating the discharges passing over the dam and for comparing the hydraulic jump behaviour observed. Measurement of water levels on the upstream of the flood regulator beyond the drawdown effect as well as on the downstream beyond the stilling basin need to be measured for correct computations. A measurement of water surface profile on either side on the left and right side be made to assess the hydraulic jump conditions.

b) Water Quality

The storage of water is one of the main objectives of the dam and therefore, monitoring of the water quality and quantity becomes imperative in the reservoir of the project. The parameters that are to be also considered in the reservoir side, is the inflow of water from the rivers and canal as it is important that no poor-quality water is led into the reservoir.

The water qualities in the dam will changes in seasons and periods due to the longitudinal profile of the river flux also both upstream and downstream of the dam. The quality of water varies periodically due to several factors such as duration of storage, the nutrient load, the depth of reservoir, inland flooding, the turbidity, temperature and sea water intrusion. The quality of water needs to be assessed for potability of the water and its use in agriculture.

c) Flow

The flow is to be considered as a major parameter in the design of dam and spillway. The flow provides the velocity from which other parameters like specific discharge, discharge, etc. can be derived.

i)Spillway

Spillway water flow rapidly varies from the crest till the jump condition is achieved. Two processes simultaneously occur in the flow over the crest, the change in the height of water and instantaneous velocity for the water level.

ii)Fish Pass

Fish passages promote and regulate safe fish migration across the dam. The measurement of Flow Parameters i.e., Discharge & Velocity etc. are very important in case of Fish Pass Channel of the Dam so that the fishes can migrate comfortably which helps sustain aquatic life. Hence, it's necessary to monitor and optimize the design based on currents.

iii)Inflow from 3 rivers and canal

The water quality and quantity are one of the major parameters that needs to be considered. The quality of water, if poor, might lead to pollution of the water in the reservoir and thus the whole water system. The quantity of water helps to determine the sectional outflow of each river/ canal to determine the actual increase in reservoir volume, due to outflow. The sensors/instruments for the hydrological parameters were given in **Table A2.322**.

Table A2.322. List of hydrological parameters and its sensors for measurement and monitoring

Hydrological Parameters	Sensors & Instruments	Data Sampling Frequency
Water Level	Automated Radar Water level Sensor	Hourly
Water Quality	Water quality Data buoy with Sensors to measure Temperature, Conductivity, Dissolved Oxygen, pH, Salinity, ORP, Turbidity, Chlorophyll-a, Blue Green Algae, Salinity and pH	Hourly
Water Flow	Ultrasonic Sensor for Flow measurement	Hourly

2.3.11 Discussion

2.3.11.1 Summary

Summary of Hydraulic, geotechnical and Structural aspects are listed as under.

A) Hydraulic Design

Hydraulic design of 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 endsill are designed to dissipate high energy turbulent flow and make flow sub-critical. Uplift pressure calculation and sheet pile length calculation has been carried out using Koshla's theory. Detailed design of all these components are covered in Volume 2 of Annexure.

B) Geotechnical Design

(i) The critical borehole locations with respect to the design is 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) Slope stability analysis is carried out for the guide bunds on both the upstream and downstream directions. Steady State and Drawdown analysis for various water levels have been analysed whichever, relevant to find a stable slope for the guide bunds.

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

C) Structural Design

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. Detailed design of the structural components is covered in Volume 2 of Annexure.

2.3.11.2 Recommendations

A) Hydraulics

Following are the important recommendations proposed,

(i) Passing a flood of 110,000 m^3/s (PMF) over the flood regulator, with a 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 infra structure 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. 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_o = 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 flood regulator recommends the stilling basin at a level of El -12.0 m. Total length of the stilling basin is equal to 50 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: thickness is as follows: 7.5 m for a length of 15 m; 5.6 m for the next 20 m and 3.2 m for the rest 15 m.

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

(viii) Pressure relief mechanism is recommended downstream of the stilling basin.

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

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

B) Geotechnical

Following are the recommendations on 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 1:2.5 is recommended to protect the bunds against waves and currents both upstream and downstream.

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

C) Structural

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.

2.3.12 Studies for Detailed Design

2.3.12.1 Field Studies

(i) After the commissioning of the flood regulator, periodic bathymetric surveys should be carried out in the downstream spill channel for any possible erosion. These surveys should be carried out mandatorily after any major flood event, and appropriate remedial measures should be taken, if required.

(ii) The ogee weir should be provided with pressure sensors at several locations on the downstream face, and should be monitored for the occurrence of cavitation, if any.
(iii) Detailed studies have been recommended to determine the oceanographic parameters on the downstream side which is utilized for the structural design.
(iv) Detailed Bathymetry investigation is recommended to ensure the founding levels of the Structural components proposed.
(v) Detailed Geotechnical investigation has to be carried out prior to construction stage to ensure the considered Safe Bearing Capacity and Other geotechnical parameters.
(v) A detailed geotechnical investigation with a minimum spacing of 50 m is recommended before detailed design of the component. The depth of investigation can be based on the type of structure that is to be constructed in the location. Special consideration is to be taken to conduct investigation on all the area over which the abutment and ogee rests.

2.2.12.2 Model Studies

(i) It is recommended that physical hydraulic model tests be conducted to confirm the safe working of the flood regulator. These physical model tests should check for the ogee weir capacity, possible occurrence of cavitation and stability of jump location for a wide-ranging flow and Downstream water level conditions.
(ii) It is recommended to carry out physical modelling studies to estimate the actual forces acting on the structural elements such as Ogee Weir, Wall Pier, Stilling Basin and Energy Dissipating Elements.

2.2.12.3 Analytical Studies

(i)Detailed Engineering drawings has to be prepared before execution.
(ii) The water levels and the wave heights that might occur in the location during and post- construction of the temporary earth works needs to be assessed and a detailed analysis on the dimensions are to be carried out before beginning the construction works.

2.3.13 Drawings

1. Flood Regulator Approach Channel and Tail Channel Details	KP/FR/0001
2. Plan of Flood Regulator	KP/FR/0002
3. Longitudinal Section of Flood Regulator	KP/FR/0003
4. Cross section of Flood Regulator	KP/FR/0004
5. Central line Drawing of Approach Channels	KP/FR/0005
6. General Arrangement of spillway piers	KP/FR/0006
7. Energy Dissipation Drawing	KP/FR/0007
8. Breast wall drawing	KP/FR/0008
9. Plan and Sections of Flank Wall	KP/FR/0009
10. Flank Wall	KP/FR/0010
11.Guide Bund	KP/FR/0011

2.3.14 Cost Estimates

The cost estimate for the flood regulator which includes the Excavation on upstream and Downstream Channel, Barrage, Approach Channel, Spill Channel, Roadway and Railway Bridge across the flood regulator portion is presented in this section.

2.3.14.1 Materials and Specifications

Materials to be used in the work shall conform to the specifications mentioned on the drawings, the requirements laid down in this section and specifications for relevant items of work covered under these Specifications.

If any material, not covered in these Specifications, is required to be used in the work, it shall conform to relevant Indian Standards, if there are any, or to the requirements consented by the Engineer.

(a) Cement

Cement to be used in the works shall be any of the following types and with prior consent of the Engineer:

(1) Ordinary Portland Cement, 33 Grade, conforming to IS:269;
(2) Rapid Hardening Portland Cement, conforming to IS: 8041;
(3) Ordinary Portland Cement, 43 Grade, conforming to IS:8112;
(4) Ordinary Portland Cement, 53 Grade, conforming to IS:12269; and
(5) Sulphate Resistant Portland Cement, conforming to IS:12330.

Cement conforming to IS:269 shall be used only after ensuring that the minimum required design strength can be achieved without exceeding the maximum permissible cement content of 500 kg/cum of concrete.

Cement conforming to IS: 8112 and IS: 12269 may be used provided the minimum cement content mentioned elsewhere from durability considerations is not reduced. From strength considerations, these cements shall be used with a certain caution as high early strengths of cement in the 1 to 28-day range can be achieved by finer grinding and higher constituent ratio of Tricalcium Silicate and Dicalcium Silicate. In such cements, the further growth of strength beyond say 4 weeks may be much lower than that traditionally expected. Therefore, further strength tests shall be carried out for 56 and 90 days to fine tune the mix design from strength considerations.

Cement conforming to IS: 12330 shall be used when sodium sulphate and magnesium sulphate are present in large enough concentration to be aggressive to concrete. It shall not be used under such conditions where concrete is exposed to risk of excessive chlorides and sulphates attack both. The recommended threshold values as per IS: 456 are sulphate concentration in excess of 0.2 per cent in soil substrata or 300 ppm (0.03per cent) in ground water. Tests to confirm actual values of sulphate concentration are essential when the structure is located near the seacoast, chemical factories, agricultural land using chemical fertilizers and sites where there are effluent discharges or where soluble sulphate bearing ground water level is high. Cement conforming to IS:12330 shall be carefully selected from strength considerations to ensure that the minimum required design strength can be achieved without exceeding the maximum permissible cement content of 500 kg/cum of concrete, respectively. Use of Fly Ash as shall not be permitted.

(b) Coarse Aggregates

For plain and reinforced cement concrete (PCC and RCC) works, coarse aggregate shall consist of clean, hard, strong, dense, non-porous and durable pieces of crushed stone, crushed gravel etc., They shall not consist pieces of disintegrated stones, soft, flaky, elongated particles, salt, alkali, vegetable matter or other deleterious materials beyond the tolerance limits specified in the relevant IS Codes. Coarse aggregate having positive alkali-silica reaction shall not be used. All coarse aggregates shall conform to IS:383 and tests for conformity shall be carried out as per IS:2386, Parts I to VIII. Marine aggregates shall not be used.

The maximum value of flakiness index for coarse aggregate shall not exceed 35 percent. The coarse aggregate shall satisfy the requirements of grading as specified in the **Table A2.326.**

Table A2.323: Requirements of Coarse Aggregates

IS Sieve Size	Percent by Weight Passing the Sieve	
	40 mm	**20 mm**
40mm	95-100	100
20mm	30-70	95-100

The size (maximum nominal) of coarse aggregates for concrete to be used in various components shall be 20mm for Reinforced Cement Concrete structures. The proportions of the various individual size of aggregates shall be so adjusted that the grading produces densest mix and the grading curve corresponds to the maximum nominal size adopted for the concrete mix.

(c) Sand / Fine Aggregates

For masonry work, sand shall conform to the requirements of IS: 2116.For plain and reinforced cement concrete (PCC and RCC) works, fine aggregate shall consist of clean, hard, strong and durable pieces of crushed stone, crushed gravel, or a suitable combination of natural sand, crushed stone or gravel. They shall not contain dust, lumps, soft or flaky, materials, mica or other deleterious materials in such quantities as to reduce the strength and durability of the concrete, or to attack the embedded steel. Motorized sand washing machines / screw type mechanical washers should be used to remove impurities from sand. Fine aggregate having positive alkali-silica reaction shall not be used. All fine aggregates shall conform to IS:383 and tests for conformity shall be carried out as per IS: 2386, (Parts I to VIII). The fineness modulus of fine aggregate shall neither be less than 2.0 nor greater than 3.5.Creek / Marine sand shall not be used in permanent works.

(d)Steel

For plain and reinforced cement concrete (PCC and RCC) or pre-stressed concrete (PSC) works, the reinforcement / un-tensioned steel, shall consist of the following grades of reinforcing bars as specified in **Table A2.327** below.

Table A2.324: Requirements of Reinforcement / Un-tensioned Steel

S. No	Grade Designation	Bar Type confirming to governing IS Specifications	Characteristic Strength f_y MPa	Elastic Modulus GP_a
1	S 240	Grade 1 Mild Steel & Medium Tensile Steel bars conforming to IS: 432 Part I Mild Steel Bar	240	200
2	2S 500	Cold twisted bars conforming to IS: 1786 High Yield Strength Deformed Bars (HYSD) / TMT bars	500	200

Other grades of bars conforming to IS:432 and IS:1786 shall not be permitted.
All the steel shall be procured only from the primary steel producers and having BIS license.

Primary steel producers are those steel (crude and / finished steel) producers using iron ore as the basic raw material / input. It therefore, includes in-house iron making followed by production of liquid steel & crude steel with / without in-house rolling. So all Integrated Steel Plants adopting BF-BOF route and major producers adopting Corex-BOF or DRI-EAF or MBF-EOF technology would fall under this category.

All reinforcing steel shall be free from loose small scales, rust and coats of paint, oil mud etc. Every bar shall be inspected before assembling on the work and defective, brittle or burnt bar shall be discarded. Cracked ends of bars shall be discarded.

(e)Water

Water used for mixing and curing shall be clean and free from injurious amounts of oils, acids, alkalis, salts, sugar, organic materials or other substances that may be deleterious to concrete or steel. Potable water is considered satisfactory for mixing concrete. Mixing and curing with sea water shall not be permitted. As a guide, the following concentrations represent the maximum permissible values:

(1) To neutralize 200 ml sample of water. using phenolphthalein as an indicator, it should not require more than 2 ml of 0.1 normal NaOH;

(2) To neutralize 200 ml sample of water, using methyl orange as an indicator, it should not require more than 10 ml of 0.1 normal HCl; and

(3) The permissible limits for solids shall be as follows when tested in accordance with IS:3025:

Permissible Limits (Max.)

Organic	:	200 mg/lit
Inorganic	:	3000 mg/lit
Sulphates (SO_4)	:	500 mg/lit
Chlorides (Cl)	:	2000 mg/lit for PCC works & 1000 mg/lit for RCC works
Suspended matter	:	2000 mg/lit

All samples of water (including potable water) shall be tested, and suitable measures taken where necessary to ensure conformity of the water to the requirements stated herein.

(1) The pH value shall not be less than 6; and

(2) In case of doubt regarding development of strength, the suitability of water for making concrete shall be ascertained by the compressive strength and initial setting time tests as specified below:

The sample of water taken for testing shall represent the water proposed to be used for concreting, due account being paid to seasonal variation. The sample shall not receive any treatment before testing other than that envisaged in the regular supply of water proposed for use in concrete. The sample shall be stored in a clean container previously rinsed out with similar water;

Average 28 days compressive strength of at least three 15cm concrete cubes prepared with water proposed to be used shall not be less than 90 percent of the average of strength of three similar concrete cubes prepared with distilled water. The cubes shall be prepared, cured and tested in accordance with the requirements of IS:516;

The initial setting time of test block made with the appropriate cement and the water proposed to be used shall not be less than 30 minutes and shall not differ by + 30 minutes from the initial setting time of control test block prepared and tested in accordance with the requirements of IS:4031; and

Water found satisfactory for mixing is also suitable for curing concrete. However, water used for curing should not produce any objectionable stain or unsightly deposit on the concrete surface. The presence of tannic acid or iron compounds is objectionable.

(f) Concrete Works

The grades of concrete shall be designated by the characteristic strength as given in **Table A2.328** below, where the characteristic strength is defined as the strength of concrete below which not more than 5 percent of the test results are expected to fall.

Table A2.325: Grades of Concrete

S. No.	Grade Designation	Specified Characteristic Compressive Strength of 150mm cubes at 28 days in MPa	Mix Ratio
1	M 25	25	1 : 1.65 : 2.92
2	M 40	40	1 : 1 : 2.5

The lowest grades of concrete in structures, corresponding minimum cementitious material contents, maximum water-cement ratios and minimum cover shall be maintained as indicated in table below based on the environmental exposure conditions

(g) Revet Mattress

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

The proposed revet mattress consists of mechanically woven double twisted hexagonal shaped wire mesh and a non-woven geotextile 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 opening. Each size may allow a variation of 5% oversize or 5% undersize, or both. Figure xx shows a typical revet mattress.

(h) Rock / Boulders

Rock has been widely used as a construction material for embankment dams. The huge quantity of material that is used in the construction of Kalpasar Dyke is Rock or Boulders. The quantity of rock estimated for the construction is approximately 160 Mm3. There are different sizes and weight of rock is required for each section. Therefore, to acquire the rock in huge quantity for the construction of kalpasar Dyke is very challenging. Rocks in current practice includes angular rock fragments as produced by quarry or occurring as talus deposits, and subangular fragments such as coarse gravel, cobbles, and boulders occurring in alluvial deposits. The main source of the Rock for Kalpasar Dyke is from Quarry. There are 15 quarries were identified in and around the Project location as shown in the figure 9. The average distance from the quarry to Kalpasar site is found to be 180 Km. The selection of the quarry is based on the rock quality. The sample of rock from each quarry is tested to know the physical and engineering properties. The quarries are chosen based on a variety of characteristics, such as water absorption, True specific gravity, slake durability, unconfined compressive strength, angle of internal friction, and cohesiveness. Based on the quality and durability scenario, the rock is said to be excellent if its mass density is greater than 2.7 t/m3 and poor if its mass density is less than 2.3 t/m3. For specification, it is desirable to limit the proportion of stone pieces with a length to thickness ratio (L/T) of greater than 3:1 to a level that is reasonable for intended use.

(i) Weep holes

Weep holes shall be provided through Retaining wall, abutments. Weep holes shall be provided with 100mm diameter pipe for structures in plain / reinforced concrete. Weep holes in the ballast wall shall be provided with 75mm diameter pipes. Weep holes shall extend through full width of the concrete with slope 1 vertical: 20 horizontaltowards draining face. Spacing of the weep holes shall generally be 1m in either direction in a staggered manner with the lowest at about 150mm above the low water level or ground level whichever is higher.

2.3.14.2 Rate Analysis

Detailed Cost estimate was carried out flood regulator. The cost estimation is carried out based on Delhi Schedule of Rates 2021 and also available Schedule of Rates of Government of Gujarat (R&B). Wherever the rates are not listed in the scheduled of rates for specific items in such case, the rates available from the estimation of Bhadbhut Barrage Project have been considered for the purpose of estimation.

2.3.14.3 Bill of Quantities

The bill of quantities includes the cost abstract of various components which includes Barrage and Flank Wall, River Floor and Guide Wall, Roadway and Railway Bridges across the Flood Regulator. The bill of quantities is summarized in the below listed **Table A2.326**, **Table A2.327**, **Table A2.328,** and **Table A2.329**.

Table A2.326: Cost Abstract for Barrage and Flank Wall

Barrage and Flank Wall					
S. No	Description	Unit	Quantity	Rate	Amount (in rupees)
1	Excavation in foundation in overburden including soil, sand, murrum, silt, gravel etc. including shoring, strutting, dewatering and depositing the excavated stuff in regular stacks or in layers with necessary levelling as directed with all lifts and lead upto 9 kms (average lead upto 4.5km) etc. complete.	cum	65,17,169.50	200.00	1,30,34,33,900.00
2	Filling in plinth with sand under floors including watering, ramming consolidating and dressing etc.complete.	cum	3,48,117.00	951.00	33,10,59,267.00
3	Providing & laying plain/ reinforced controlled cement concrete of specified grade with cement, sand and coarse aggregates, admixtures in recommended proportions as specified to accelerate/ retard setting of concrete including centering, shuttering,	cum	52,217.55	11,335.00	59,18,85,929.25

	weigh batching using automatic mixing plant, transporting, placing, vibrating, smooth finishing, curing etc. complete with all lead and lift excluding cost of reinforcement. (Including dewatering) M-25 (Cement level 340kg)				
4	Providing, fabricating and placing in position Corrosion Resistant Steel (CRS) fe 500D (with treatment under IS 9077-1979) for RCC structures including cleaning, straightening, cutting, bending, hooking, lapping / welding joints/rebar coupler (for joining rebar above 32mm dia) wherever required, tying with 1.25 mm diameter soft annealed steel wire, including cost of all materials, machinery, labour etc., complete with average lead upto 2 km and all lifts.	MT	2,38,307.37	75,068	17,88,92,57,538.56
5	Providing & laying plain/ reinforced controlled cement concrete of specified grade with cement, sand and coarse aggregates, admixtures in recommended proportions as specified to accelerate/ retard setting of concrete including cantering, shuttering, weigh batching using automatic mixing	cum	16,70,148.20	12,380.00	20,67,64,34,716.00

	plant, transporting, placing, vibrating, smooth finishing, curing etc. complete with all lead and lift excluding cost of reinforcement. (Including dewatering) Precooled M-40 (Cement level 410kg)				
6	Providing & laying plain/ reinforced controlled cement concrete of specified grade with cement, sand and coarse aggregates, admixtures in recommended proportions as specified to accelerate/ retard setting of concrete including cantering, shuttering, weigh batching using automatic mixing plant, transporting, placing, vibrating, smooth finishing, curing etc. complete with all lead and lift excluding cost of reinforcement. (Including dewatering) Precooled M-25 (Cement level 330kg) 1) Flank wall	cum	2,85,355.00	11,335.00	3,23,44,98,925.00
7	Providing and laying in position 75 mm dia P.V.C. Pipe for weep holes with non-corroding jali in flank wall, guide wall etc. complete.	RMT	1,430.00	118.66	1,69,683.80
8	Providing and Fixing PVC non-return valve of 75mm of approved quality as specified	No	286.00	1,710.00	4,89,060.00

9	Back filling the foundation trenches around the structures etc with selected excavated stuff including watering, ramming, compacting etc. complete.	cum	2,85,350.00	443.00	12,64,10,050.00
10	Providing and fixing in position 25 mm thick bitumen impregnated fiber board including cost of primer, sealing compound Grade-A in expansion joints as shown in drawing etc. complete.	LS			2,00,00,000.00
11	Providing and fixing in position steel sheet piles of specified size, shape and length in river bed in diversion structure including cost of steel sheet piles and all supports for positioning the sheet pile, transporting, loading, unloading, fixing & extracting in all phases and final extraction &transporation upto disposal yard in position as directed by Engineer-in-Charge complete.	MT	12,576.14	49,500.00	62,25,18,831.00
Total					44,79,61,57,900.61

Table A2.327: Cost Abstract for River Floor and Guide Wall

River Floor and Guide Wall					
S. No	Description	Unit	Quantity	Rate	Amount (in rupees)
1	Excavation in foundation in overburden including soil, sand, murrum, silt, gravel etc. including shoring, strutting, dewatering and depositing the excavated stuff in regular stacks or in layers with necessary levelling as directed with all lifts and lead	cum	39,55,90,787.50	200.00	79,11,81,57,500.00
2	Filling in plinth with sand under floors including watering, ramming consolidating and dressing etc. complete.	cum	26,83,897.50	951.00	2,55,23,86,522.50
3	Providing and laying stone spall layer of specified thickness with quarry spall provided over the entire prepared compacted base surface below precast cement concrete block in U/S & D/S side river protection work as shown in drawing, including all lead, lift, watering and compacting complete as per drawing and	cum	27,05,208.00	489.60	1,32,44,69,836.80

	technical specifications as directed by engineer-in-charge. (600 mm Thick Stone below concrete blocks)				
4	Providing and fixing precast cement concrete blocks of size 1.5x1.5x0.9m size of M-25 grade using cement content 360kg/cum as per approved design mix (stone aggregate 20 mm nominal size), in river training work in U/S side, including cost of reinforcement for hook, centering, shuttering & finishing, weigh batching using automatic mixing plant, vibrating, dewatering, curing, transporting, placing & preparation of sub grade using mechanical means etc. complete as directed by Engineer- in - Charge. (Transportation, placing with mechanical means)	Cum	40,08,594.83	11335.00	45,43,74,22,341.38
5	Providing, fabricating and placing in position Corrosion Resistant Steel (CRS) fe 500D (with treatment under IS 9077-1979) for RCC	MT	60,128.92	75,068.00	4,51,37,57,944.85

	structures including cleaning, straightening, cutting, bending, hooking, lapping / welding joints/rebar coupler (for joining rebar above 32mm dia) wherever required, tying with 1.25 mm diameter soft annealed steel wire, including cost of all materials, machinery, labour etc., complete with average lead upto 2 km and all lifts.				
6	Providing & making Gabion structure with Mechanically Woven Double Twisted Hexagonal Shaped wire mesh Gabion Boxes as per IS 16014:2012, MORTH Clause 2500, of specified size, Mesh Type 10x12 (D=100 mm with tolerance of ±2%), Zinc+PVC coated, Mesh wire diameter 2.7/3.7mm (ID/OD), mechanically edged/selvedged with partitions at every 1m interval and shall have minimum 10 numbers of openings per meter of mesh perpendicular to twist, tying with lacing wire of diameter 2.2/3.2mm(ID/OD),	cum	9,35,970.00	2039.85	1,90,92,38,404.50

	supplied @3% by weight of Gabion boxes, filled with 30 to 50 kg trap rubble (approximate 3 ton), including packing, interlocking of stones and fusing top of gabion & tying to each other & laying to the required line, level slope as per drawing with all lead and lift etc. complete as directed by Engineer-in-charge. (A) 500 mm thick Gabion Mattress on inclined face of Guide bund				
7	Back filling the foundation trenches around the structures etc with selected excavated stuff including watering, ramming, compacting etc. complete.	Cum	98,65,425.00	443.00	4,37,03,83,275.00
Total					**1,39,22,58,15,825.02**

Table A2.328: Cost Abstract for Railway Bridge across Flood Regulator portion

Railway Bridge across flood regulator portion					
S. No	**Description of work**	**Unit**	**Quantity**	**Rate**	**Amount**
1	Excavation for Structures (Earth work in excavation of foundation of structures as per MoRTH technical specification, including setting out, construction of shoring and bracing, removal of stumps and other deleterious matter, dressing of sides and bottom and backfilling with approved material.)	Cum	45,494	85	38,67,029
2	Plain Cement Concrete in levelling course excluding reinforcement complete as per drawings and Technical Specifications Sections1500, 1700 and 2100. M15 grade	Cum	2,967	5,902	1,75,11,411
3	Installation of Pile: Driving of Piles including bentonite	Rmt	69,300	4,816	33,37,48,800
4	Reinforced Cement Concrete in Foundation excluding reinforcement complete as per MoRTH Technical Specifications Sections 1500, 1700 and 2200. M35 grade foundation	Cum	1,13,981	6,294	71,73,94,400
5	Reinforced Cement Concrete in Substructure excluding reinforcement complete as per MoRTH Technical Specifications Sections 1500, 1700 and 2200. M35 grade Substructure	Cum	2,07,525	6,591	1,36,77,95,891

Railway Bridge across flood regulator portion					
S. No	**Description of work**	**Unit**	**Quantity**	**Rate**	**Amount**
6	Reinforced Cement Concrete in Superstructure excluding reinforcement complete as per MoRTH Technical Specifications Sections 1500, 1700 and 2200. M35 grade Superstructure	Cum	3,480	7,678	2,67,19,440
7	Reinforced Cement Concrete in Superstructure excluding reinforcement complete as per MoRTH Technical Specifications Sections 1500, 1700 and 2200. M45 grade Superstructure	Cum	13,340	8,066	10,75,99,230
8	Reinforced Cement Concrete in Superstructure excluding reinforcement complete as per MoRTH Technical Specifications Sections 1500, 1700 and 2200. M55 grade Superstructure	Cum	25,585	9,975	25,52,09,976
9	Strip Seal Expansion Joint (Providing and laying of a strip seal expansion joint catering to maximum horizontal movement upto 70 mm, complete as per approved drawings and standard specifications to be installed by the manufacturer/supplier or their authorised representative ensuring compliance to the manufacturer's instructions for installation.)	Rmt	2,505	4,225	1,05,82,780

Railway Bridge across flood regulator portion					
S. No	**Description of work**	**Unit**	**Quantity**	**Rate**	**Amount**
10	Providing and laying wearing Coat of 65mm thick with Mastic Asphalt 25mm & Bituminous Concrete 40mm with paving grade bitumen meeting the requirements given in table 500-29, prepared by using mastic cooker and laid to required level and slope.	Cum	-	5,615	-
11	PCC Wearing coat over deck slab of M15 Grade for Railway Bridge	Cum	261	5,902	15,42,547
12	Providing, cutting, bending and fixing TMT/HYSD bar reinforcement in reinforced concrete structures complete as MORTH Technical Specification Section 1600. (Fe 500 Grade)	MT	49,403	1,18,239	5,84,13,78,461
13	Elastomeric Bearing	Cum	60	5,20,000	3,14,49,600
14	Pin Bearing	Nos	400	60,000	2,40,00,000
15	Metallic Guided Bearing	Nos	400	60,000	2,40,00,000
16	High tensile steel wires/strands including all accessories for stressing, stressing operations and grouting complete as MORTH Technical Specifications	MT	2,112	1,66,847	35,24,47,603
Total Amount (Rs.)					**9,11,52,47,167**

Table A2.329: Cost Abstract for Roadway Bridge across Flood Regulator portion

Roadway Bridge across flood regulator portion					
S. No	**Description of work**	**Unit**	**Quantity**	**Rate**	**Amount**
1	Excavation for Structures (Earth work in excavation of foundation of structures as per MoRTH technical specification, including setting out, construction of shoring and bracing, removal of stumps and other deleterious matter, dressing of sides and bottom and backfilling with approved material.)	Cum	1,12,580	85	95,69,329
2	Plain Cement Concrete in levelling course excluding reinforcement complete as per drawings and Technical Specifications Sections1500, 1700 and 2100. M15 grade	Cum	7,342	5,902	4,33,33,643
3	Installation of Pile: Driving of Piles including bentonite	Rmt	1,80,180	4,816	86,77,46,880
4	Reinforced Cement Concrete in Foundation excluding reinforcement complete as per MoRTH Technical Specifications Sections 1500, 1700 and 2200. M35 grade foundation	Cum	2,91,886	6,294	1,83,71,28,344
5	Reinforced Cement Concrete in Substructure excluding reinforcement complete as per MoRTH Technical Specifications Sections 1500, 1700 and 2200. M35 grade Substructure	Cum	5,15,305	6,591	3,39,63,74,596
6	Reinforced Cement Concrete in Superstructure excluding reinforcement	Cum	72,874	7,678	55,95,27,800

Roadway Bridge across flood regulator portion					
S. No	**Description of work**	**Unit**	**Quantity**	**Rate**	**Amount**
	complete as per MoRTH Technical Specifications Sections 1500, 1700 and 2200. M35 grade Superstructure				
7	Reinforced Cement Concrete in Superstructure excluding reinforcement complete as per MoRTHT echnical Specifications Sections 1500, 1700 and 2200. M45 grade Superstructure	Cum	56,893	8,066	45,88,95,550
8	Reinforced Cement Concrete in Superstructure excluding reinforcement complete as per MoRTH Technical Specifications Sections 1500, 1700 and 2200. M55 grade Superstructure	Cum	-	9,975	-
9	Strip Seal Expansion Joint (Providing and laying of a strip seal expansion joint catering to maximum horizontal movement upto 70 mm, complete as per approved drawings and standard specifications to be installed by the manufacturer/supplier or their authorised representative ensuring compliance to the manufacturer's instructions for installation.)	Rmt	9,747	4,225	4,11,78,963
10	Providing and laying wearing Coat of 65mm thick with Mastic Asphalt 25mm & Bituminous Concrete 40mm with paving	Cum	627	5,615	35,22,250

Roadway Bridge across flood regulator portion					
S. No	**Description of work**	**Unit**	**Quantity**	**Rate**	**Amount**
	grade bitumen meeting the requirements given in table 500-29, prepared by using mastic cooker and laid to required level and slope.				
11	PCC Wearing coat over deck slab of M15 Grade for Railway Bridge	Cum	-	5,902	-
12	Providing, cutting, bending and fixing TMT/HYSD bar reinforcement in reinforced concrete structures complete as MORTH Technical Specification Section 1600. (Fe 500 Grade)	MT	1,24,828	1,18,239	14,75,95,29,095
13	Elastomeric Bearing	Cum	125	5,20,000	6,48,54,400
14	Pin Bearing	Nos	-	60,000	-
15	Metallic Guided Bearing	Nos	-	60,000	-
16	High tensile steel wires/strands including all accessories for stressing, stressing operations and grouting complete as MORTH Technical Specifications	MT	2,724	1,66,847	45,44,52,853
Total Amount (Rs.)					**22,49,61,13,704**

2.3.14.4 Costing

The cost estimate for Flood Regulator portion has been carried out based on the Delhi schedule of rates 2021 and Gujarat Schedule of Rates. The cost estimate includes the various components such as Excavation in Upstream and Downstream side of the Flood Regulator, Approach Channel, Spill Channel, Flood Regulator and Bridge over Flood Regulator. The costing of this section is summarized in **Table A2.330** and it is estimated to be around Rs **2, 15, 63, 33, 34, 595 crores.**

Table A2.330: Cost Abstract for Flood Regulator portion

S. No	Description	Amount in INR
1	Excavation in Upstream Side	17,38,75,28,600
2	Excavation in Downstream Side	63,03,40,62,800
3	Approach Channel	53,50,81,08,598
4	Spill Channel	14,17,85,77,465
5	Barrage	35,91,36,96,261
6	Bridge Over Barrage	31,61,13,60,871
	Total Rs.	**2,15,63,33,34,595**

Geotechnical Interpretative Report

1. Preamble:

Geo Marine Consultants Pvt. Ltd, Chennai carried out geotechnical investigations on the Eastern side of L-3 alignment of Kalpasar Dam in Gujarat, India and submitted their report titled "Geotechnical Investigation Report" dated, 14th February 2023. Twenty-two bore holes were carried out up to a depth ranging from 20 m to 40 m. In addition, 13 Electric Cone Penetration Tests (ECPT) were carried out covering the approach, spill channel and main flood regulator locations (Figure 1).

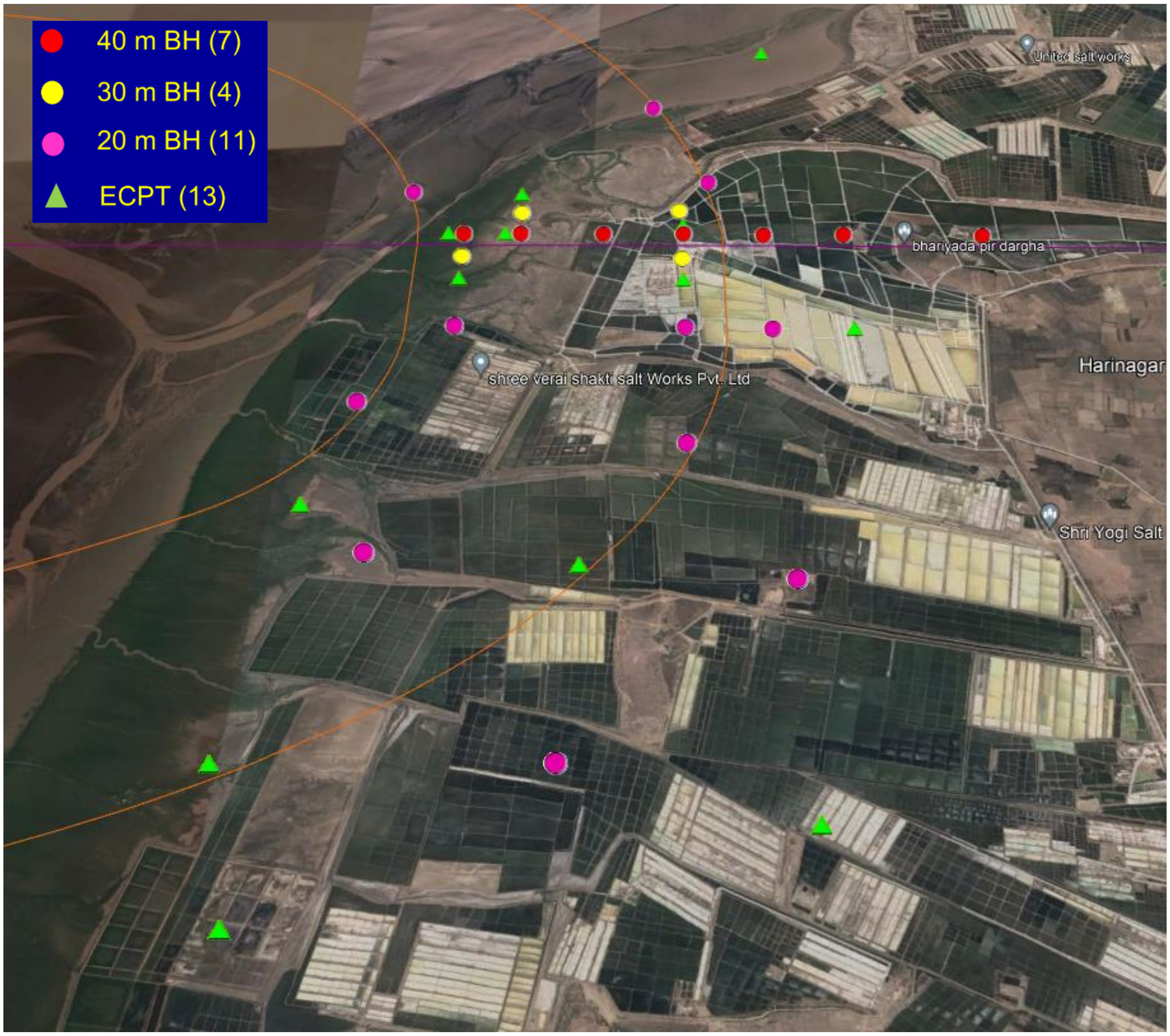


Figure 1 Google Earth image showing locations of boreholes and ECPTs

Both disturbed and undisturbed soil samples were collected. Field identification and laboratory tests including natural moisture content, Atterberg's limits, Grain Size Analysis, Relative density, consistency, consolidation, triaxial tests and chemical analysis tests on soil and water samples (pH, chlorides and sulphates) were carried out.

Depth of ground water table varies from 0.6 m to 2 m below existing ground level.

The reported spoil strata consist of very soft marine clay (depth ranging from 3 m to 12 m below the seabed). The top soil is too soft such that the testing equipment got overturned at many locations. Due to this, locations of few boreholes / ECPTs had to be shifted. Table 1 gives the details of boreholes and ECPTs carried out in flood regulator, approach and spill channels.

Table 1 Details of field tests

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
ECPT's	T1, T2, T9	T5	T3, T6, T11, T12, T13

This report deals with

- Observations and findings from the geotechnical investigation report.
- Interpretation of strength parameters from field and lab test results.
- Interpretation of shear parameters from ECPTs
- Comparison of soil strata from nearby ECPTs and SPTs.
- Recommendation of engineering soil parameters for geotechnical design.

2. Observations from the geotechnical report:

i. Length of approach channel : 10 km

Length of spill channel : 10 km

The borehole data available in approach channel is upto 2 km from the flood regulator. Similarly, the borehole data available in spill channel is upto 4.5 km from the flood regulator.

Four boreholes are available in the location of flood regulator. All the four have been used in the design of cellular coffer dam.

For the preliminary design, it can be assumed that the same soil profile continues for entire length of the spill channel beyond 4.5 km. Detailed investigations are to be carried out covering the entire length for the final design.

3. Interpretation of strength parameters from field and lab test results:

Based on SPT N values, cohesion values for cohesive soils are interpretated using plasticity index from CIRIA correlations. Angle of internal friction for cohesionless soils are obtained using SPT N from IS 6403-1981. The interpretated soil parameters and obtained parameters from laboratory tests are compared and presented in Enclosure 1.

4. Interpretation of shear parameters from ECPT results

Based on the type of soil strata, the cone penetration values are interpretated to obtain cohesion and SPT N values using correlation from IS 2911: 2010 (Part 1/Sec 1). The interpretated SPT N values are then correlated to obtain friction angle using IS 6403-1981. The obtained shear parameters are given in Enclosure 2.

5. Comparison of soil strata - Nearby ECPTs and Boreholes

Nearby boreholes and ECPTs were identified based on the location plan. The minimum distance between nearby boreholes and ECPTs is 90 m, yet the soil strata is compared to get an overview of variation of strata conditions (Enclosure 3).

6. Shear Parameters used for design:

Design of Cellular Coffer Dam:

Borehole C1			
Depth(m)	**Strata**	**Shear Parameters**	
		Cohesion(kPa)	**Φ (degree)**
0 – 1.5	Soft clay	4.5	-
1.5 – 7.5		9	-
7.5 – 13.5	Stiff clay	123	-
13.5 – 27		101	-
27 – 40	Dense sand	-	40

Borehole C2			
Depth(m)	**Strata**	**Shear Parameters**	
		Cohesion(kPa)	**Φ (degree)**
0 – 6	Soft clay	5	-
6 – 7.5		13.5	-
7.5 – 10.5	Silt	-	30
10.5 – 21	Stiff clay	98	-
21 – 25.5		114	-
25.5 – 31.5		107	-
31.5 – 33		171	-
33 – 36		240	-
36 – 37.5		302	-
37.5 – 40	Dense sand	-	40

Borehole C3			
Depth(m)	Strata	Shear Parameters	
		Cohesion(kPa)	Φ (degree)
0 – 1.5	Stiff clay	31.5	-
1.5 – 7.5	Soft clay	5	-
7.5 – 10.5	Stiff clay	64.5	-
10.5 – 13.5		63	-
13.5 – 16.5		121	-
16.5 – 30	Silt	-	30
30 – 40	Dense sand	-	40

Borehole C7			
Depth(m)	Strata	Shear Parameters	
		Cohesion(kPa)	Φ (degree)
0 – 4.5	Soft clay	5	-
4.5 – 10.5	Stiff clay	75	-
10.5 – 16.5		70	-
16.5 – 21		110	-
21 – 22.5		230	-
22.5 – 28.5		90	-
28.5 - 40	Dense sand	-	40

Spill Channel Design:

Borehole D4			
Depth(m)	Strata	Shear Parameters	
		Cohesion(kPa)	Φ (degree)
0 – 6.5	Soft clay	0	-
6.5 – 9.5	Stiff clay	62	-
9.5 – 15.5		135	-
15.5 – 17		90	-
17 – 20		120	-

Borehole D8			
Depth(m)	Strata	Shear Parameters	
		Cohesion(kPa)	Φ (degree)
0 – 0.5	Soft clay	0	-
0.5 – 3.5		5	-
3.5 – 6.5		0	-
6.5 – 9.5		5	-
9.5 – 17.5	Stiff clay	95	-
17.5 – 20		140	-

ENCLOSURE 1

INTERPRETATION OF STRENGTH PARAMETERS FROM FIELD AND LAB TEST RESULTS

Borehole – A1						
Depth (m)	**Strata**	**N value**	**Reported**		**Interpreted**	
			Cohesion (kPa)	**Φ (degree)**	**Cohesion (kPa)**	**Φ (degree)**
0 - 6	Very soft clay	0			0	0
6 – 7.5	Silty sand	19	2.88	31.45	-	30
7.5 – 15	Stiff clay	18	110	7.29	100	-
15 – 19.5	Very stiff clay	27			150	-
19.5 – 21	Clayey silt	27			-	32
21 - 24	Very stiff clay	22			120	-
24 - 30	Hard clay	35			165	-

Borehole – A2						
Depth (m)	**Strata**	**N value**	**Reported**		**Interpreted**	
			Cohesion (kPa)	**Φ (degree)**	**Cohesion (kPa)**	**Φ (degree)**
0 - 7.5	Very soft clay	0			0	0
7.5 – 9	Clayey silt	9			-	29
9 – 13.5	Stiff Clay	9, 11			42	-
13. 5 - 16.5		19			95	-
16.5 – 21	Very stiff clay	23	75	5.36	105	-
21 – 27		28			140	-
27 - 30	Dense Sand	47			-	40

Borehole – A3						
Depth (m)	**Strata**	**N value**	**Reported**		**Interpreted**	
			Cohesion (kPa)	**Φ (degree)**	**Cohesion (kPa)**	**Φ (degree)**
0 -1.1	Soft clay	1			6.61	-
1.1 – 4.5	Very soft clay	0			0	0
4.5 – 7.5	Soft clay	1			5.7	-
7.5 – 9	Soft clay	1			5.55	-
9 – 12	Medium Stiff clay	5	109	3.18	30	-
12 – 13.3	Very Stiff clay	31			205	-
13.3 – 15.2	Silty sand	31	3.25	34.35	-	36
15.2 – 21	Very Stiff clay	25	135	4.82	140	-
21 – 24	Hard clay	30			165	-
24 – 30		42			250	-
30 – 30.5	Dense sand	59			-	42

Borehole – A4						
Depth (m)	**Strata**	**N value**	**Reported**		**Interpreted**	
			Cohesion (kPa)	**Φ (degree)**	**Cohesion (kPa)**	**Φ (degree)**
0 – 1.5	Soft clay	1			4.5	-
1.5 – 10.5	Very soft clay	0			0	-
10.5 – 13.5	Stiff clay	14			70	-
13.5 – 16.5	Very Stiff clay	21	115	4.47	130	-
16.5 – 19.5		21			130	-
19.5 – 22.5		18	108	3.94	80	-
22.5 – 28.5		22			100	-
28.5 - 30	Hard clay	32			155	-

Borehole – C1						
Depth (m)	Strata	N value	Reported		Interpreted	
			Cohesion (kPa)	Φ (degree)	Cohesion (kPa)	Φ (degree)
0 – 4.5	Soft clay	1			6	-
4.5 – 7.5	Clayey silt	2			-	20
7.5 – 9	Sand	22	2.43	35.85	-	32
9 – 10.5	Very Stiff clay	22			100	-
10.5 – 13.5	Hard clay	32			145	-
13.5 – 16.5	Stiff clay	18	120	4.29	80	-
16.5 – 19.5	Very Stiff clay	25	131	4.15	135	-
19.5 – 27		23			110	-
27 - 40	Dense sand	>50			-	42

Borehole – C2						
Depth (m)	**Strata**	**N value**	**Reported**		**Interpreted**	
			Cohesion 0(kPa)	**Φ (degree)**	**Cohesion (kPa)**	**Φ (degree)**
0 – 6	Very soft clay	0			0	0
6 – 7.5	Soft clay	3			18	-
7.5 – 9	Clayey silt	3			-	21
9 – 10.5	Silty sand	20	0.7	31.98	-	32
10.5 – 15	Very Stiff clay	20	103	4.57	90	-
15 – 18		20			135	-
18 - 21		25	138	5.29	145	-
21 – 24	Hard clay	31			160	-
24 – 28.5	Very Stiff clay	27			125	-
28.5 – 33		28			125	-
33 – 36	Dense sand	43			-	45
36 - 40	Hard clay	51			340	-

Borehole – C3						
Depth (m)	**Strata**	**N value**	**Reported**		**Interpreted**	
			Cohesion (kPa)	**Φ (degree)**	**Cohesion (kPa)**	**Φ (degree)**
0 – 1.5	Medium stiff clay	7			45	-
1.5 – 10.5	Very soft clay	0			0	-
10.5 – 13.5	Stiff clay	14			62	-
13.5 – 22.5	Very Stiff clay	27	141	4.15	150	-
22.5 – 30	Hard clay	33			220	-
30 – 34.5	Dense sand	44, 36			-	39
34.5 - 40		>50			-	41

Borehole – C4						
Depth (m)	**Strata**	**N value**	**Reported**		**Interpreted**	
			Cohesion (kPa)	**Φ (degree)**	**Cohesion (kPa)**	**Φ (degree)**
0 – 3	Very soft clay	0			0	-
3 – 4	Soft clay	1			6.61	-
4 – 9		2			11	-
9 – 12	Clay	5			33	-
12 – 18	Stiff clay	16			110	-
18 – 20	Very Stiff clay	24	99	3.05	110	-
21 – 24	Stiff clay	12	119	3.62	60	-
24 – 25.5	Dense sand	31			-	36
25.5 - 41		>50			-	42

Borehole – C5						
Depth (m)	**Strata**	**N value**	**Reported**		**Interpreted**	
			Cohesion (kPa)	**Φ (degree)**	**Cohesion (kPa)**	**Φ (degree)**
0 – 3	Very soft clay	0			0	-
3 – 9	Soft clay	1			5.35	-
9 – 12	Soft clay	2	27	2.02	10	-
12 – 15	Stiff clay	8			55	-
15 – 17	Very Stiff clay	20			130	-
17 – 21	Stiff clay	6	78	5.14	40	-
21 - 24	Very Stiff clay	13			90	-
24 – 30	Dense sand	42	4.9	35	-	39
30 - 40		>50			-	44

Borehole – C6						
Depth (m)	**Strata**	**N value**	**Reported**		**Interpreted**	
			Cohesion (kPa)	**Φ (degree)**	**Cohesion (kPa)**	**Φ (degree)**
0 – 3.5	Soft clay	2			10	-
3.5 – 7.5		3			14	-
7.5 – 13.5	Stiff clay	12			55	-
13.5 – 16.5		19	29	3.18	130	-
16.5 – 22.5	Very Stiff clay	29	112	5.39	185	-
22. 5 -27		18			120	-
27 – 33	Hard clay	36			162	-
>33	Dense sand	>100			-	44

Borehole – C7						
Depth (m)	**Strata**	**N value**	**Reported**		**Interpreted**	
			Cohesion (kPa)	**Φ (degree)**	**Cohesion (kPa)**	**Φ (degree)**
0 – 4.5	Very soft clay	0	4.5	34.1	0	-
4.5 – 9	Clayey sand	14			-	31
9 – 10.5	Stiff clay	18			80	-
10.5 – 13.5		9	57	5.24	40	-
13.5 – 16.5	Very Stiff clay	18			90	-
16.5 – 22.5		20			90	-
22.5 – 25.5		17			81	-
25.5 – 30		22			100	-
30 – 37.5	Dense sand	>50			-	42
37.5 - 40		>100			-	44

Borehole – D1						
Depth (m)	**Strata**	**N value**	**Reported**		**Interpreted**	
			Cohesion (kPa)	**Φ (degree)**	**Cohesion (kPa)**	**Φ (degree)**
0 – 7.5	Very soft clay	0			0	-
7.5 – 9	Clayey silt	0	135	4.29	0	0
9 – 12	Silty sand	7			-	29
12 – 15	Very Stiff clay	23			105	-
15 – 18		27	135	5	120	-
18 – 20.5		22			115	-

Borehole – D2						
Depth (m)	**Strata**	**N value**	**Reported**		**Interpreted**	
			Cohesion (kPa)	**Φ (degree)**	**Cohesion (kPa)**	**Φ (degree)**
0 – 12	Very soft clay	0	32	4.45	0	-
12 - 15	Stiff clay	12			55	-
15 – 16.5	Hard clay	33			150	-
16.5 – 18	Very stiff clay	23			105	-
18 – 19.5	Hard clay	37			170	-
19.5 – 21.5		31			150	-

Borehole – D3						
Depth (m)	**Strata**	**N value**	**Reported**		**Interpreted**	
			Cohesion (kPa)	**Φ (degree)**	**Cohesion (kPa)**	**Φ (degree)**
0 – 4.5	Very soft clay	0			0	-
4.5 – 10.5		1	45	4.68	5	-
10.5 – 16.5	Very stiff clay	21			95	-
16.5 – 18		30			140	-
18 – 19.5		25			115	-
19.5 - 21	Hard clay	39			180	-

Borehole – D4						
Depth (m)	**Strata**	**N value**	**Reported**		**Interpreted**	
			Cohesion (kPa)	**Φ (degree)**	**Cohesion (kPa)**	**Φ (degree)**
0 – 7.5	Very soft clay	0	32	4.51	0	-
7.5 – 10.5	Stiff clay	14			65	-
10.5 – 13.5	Very stiff clay	27			140	-
13.5 – 16.5	Hard clay	31			190	-
16.5 – 18	Very stiff clay	20			95	-
18 – 19.5	Hard clay	37			190	-
19.5 - 21	Very stiff clay	22			110	-

Borehole – D5						
Depth (m)	**Strata**	**N value**	**Reported**		**Interpreted**	
			Cohesion (kPa)	**Φ (degree)**	**Cohesion (kPa)**	**Φ (degree)**
0 – 6	Very soft clay	0	60	7.83	0	-
6 – 9	Very loose Silty sand	0			-	0
9 – 10.5	Dense Silty sand	44	2.42	33.8	-	40
10.5 – 13.5		31			-	36
13.5 – 15	Very stiff clay	26			120	-
15 – 19.5	Hard clay	40			180	-
19.5 - 21	Very stiff clay	24			110	-

Borehole – D6						
Depth (m)	**Strata**	**N value**	**Reported**		**Interpreted**	
			Cohesion (kPa)	**Φ (degree)**	**Cohesion (kPa)**	**Φ (degree)**
0 – 4.5	Very soft clay	0			0	-
4.5 – 6	Clayey sand	0			-	0
6 – 9	Very soft clay	0			0	-
9 – 15	Very stiff clay	16	142	2.58	80	-
15 – 18		27			160	-
18 - 20		28	133	3.6	140	-

Borehole – D7						
Depth (m)	**Strata**	**N value**	**Reported**		**Interpreted**	
			Cohesion (kPa)	**Φ (degree)**	**Cohesion (kPa)**	**Φ (degree)**
0 – 4.5	Very soft clay	0			0	-
4.5 – 6	Silty sand	7			-	29
6 – 10.5	Stiff clay	7			45	-
10.5 - 13.5	Very stiff clay	24	124	3.72	125	-
13. 5 - 16.5		25			130	-
16.5 – 20		27	180	7.58	140	-

Borehole – D8						
Depth (m)	**Strata**	**N value**	**Reported**		**Interpreted**	
			Cohesion (kPa)	**Φ (degree)**	**Cohesion (kPa)**	**Φ (degree)**
0 – 1.5	Very soft clay	0			0	-
1.5 – 10.5	Very soft clay	1			5.55	-
10.5 – 13.5		20			95	-
13.5 – 16.5	Very stiff clay	22			100	-
16.5 – 19.5		19	145	4.43	90	-
19.5 – 21.5	Hard clay	31			150	-

Borehole – D10						
Depth (m)	Strata	N value	Reported		Interpreted	
			Cohesion (kPa)	Φ (degree)	Cohesion (kPa)	Φ (degree)
0 – 1.5	Very soft clay	2			13	-
1.5 – 4.5		1			5.5	-
4.5 – 7.5		0			0	-
7.5 – 10.5	Soft clay	2			11	-
10.5 – 16.5	Very stiff clay	18	112	4.57	94	-
16.5 - 20		25			130	-

Borehole – D11						
Depth (m)	**Strata**	**N value**	**Reported**		**Interpreted**	
			Cohesion (kPa)	**Φ (degree)**	**Cohesion (kPa)**	**Φ (degree)**
0 – 6	Very soft clay	0			0	-
6 – 7.5	Silty sand	7			-	28
7.5 – 9	Stiff clay	7	74	4.29	35	-
9 – 15	Very stiff clay	22			105	-
15 – 18		27	152	4.82	145	-
18 - 20		28			170	-

Borehole – D12						
Depth (m)	**Strata**	**N value**	**Reported**		**Interpreted**	
			Cohesion (kPa)	**Φ (degree)**	**Cohesion (kPa)**	**Φ (degree)**
0.5 – 3	Very soft clay	0			0	-
3 – 6		2			9	-
6 – 9	Stiff clay	11			70	-
9 – 12	Very stiff clay	21	52	8.73	110	-
12 – 13.5	Silt	25			-	32
13.5 - 18	Very stiff clay	25	95	3.58	165	-
18 - 20	Hard clay	32			160	-

ENCLOSURE 2

INTERPRETATION OF SHEAR PARAMETERS FROM ECPT RESULTS

ECPT – T1					
Depth (m)	**Soil Strata**	**Cone resistance (qc) [kPa]**	**N value**	**Cohesion (kPa)**	**Φ (degree)**
0 – 1	Very stiff fine grained	500	3	20	-
1 – 2	Clayey silt to Silty clay	500	3	20	-
2 – 3		1000	6	35	-
3 – 4		500	5	30	-
4 – 5	Sands	2000	6	-	25
5 - 6	Clayey silt to Silty clay	1100	5	30	-
6 – 7	Sands	8000	23	-	32
7 – 8	Clay to Silty clay	2500	14	80	-
8 – 9		3000	17	95	-
9 – 10		3000	17	75	-
10 – 11		3000	17	75	-
11 – 12		3100	18	80	-
12 – 13		3500	20	90	-
13 – 14		3000	17	80	-
14 - 15		4000	23	105	-

ECPT – T2					
Depth (m)	**Soil Strata**	**Cone resistance (qc) [kPa]**	**N value**	**Cohesion (kPa)**	**Φ (degree)**
0 – 1	Very stiff fine grained	1000	6	40	-
1 – 2		1500	9	60	-
2 – 3	Clay to silty clay	500	3	20	-
3 – 4	Organic soil - peat	500	3	20	-
4 – 5	Clay to silty clay	700	4	25	-
5 - 6		300	2	15	-
6 – 7		500	3	15	-
7 – 8		1000	6	30	-
8 – 9		2200	13	60	-
9 – 10		2500	14	65	-
10 – 11		2500	14	65	-
11 – 11.4		2500	14	65	-

ECPT – T3					
Depth (m)	**Soil Strata**	**Cone resistance (qc) [kPa]**	**N value**	**Cohesion (kPa)**	**Φ (degree)**
0 – 1	Stiff sand to clayey sand	1000	6	40	-
1 – 2	Clay to silty clay	800	5	35	-
2 – 3	Clayey silt to silty clay	1200	7	40	-
3 – 4	Clay to silty clay	1100	6	30	-
4 – 5		700	4	20	-
5 - 6		1000	6	35	-
6 – 7		1100	6	25	-
7 – 8		1600	9	40	-
8 – 9		2100	12	70	-
9 – 10		3000	17	95	-
10 – 11		2400	14	80	-
11 – 12		2000	11	65	-
12 – 13		4000	23	150	-
13 – 14		4100	23	150	-
14 - 15		4500	26	170	-
15 – 15.7		4000	23	105	-

ECPT – T4					
Depth (m)	**Soil Strata**	**Cone resistance (qc) [kPa]**	**N value**	**Cohesion (kPa)**	**Φ (degree)**
0 – 1	Very stiff fine grained	2100	12	70	-
1 – 2	Clay to silty clay	1000	6	35	-
2 – 3	Sand mixtures	1000	4	-	26
3 – 4	Clayey to silty clay	1000	6	35	-
4 – 5	Clay to silty clay	1000	6	35	-
5 - 6		2100	12	75	-
6 – 7		3100	18	95	-
7 – 8		3200	18	95	-
8 – 9		3500	20	105	-
9 – 10		3500	20	105	-
10 – 11		7200	41	210	-
11 – 11.4	Very stiff fine grained	7200	41	210	-

ECPT – T5					
Depth (m)	**Soil Strata**	**Cone resistance (qc) [kPa]**	**N value**	**Cohesion (kPa)**	**Φ (degree)**
0 – 1	Very stiff fine grained	2500	14	75	-
1 – 2	Clay to silty clay	1000	6	30	-
2 – 3		1000	6	35	-
3 – 4		1100	7	40	-
4 – 5		1500	9	50	-
5 - 6		2000	11	75	-
6 – 7	Sands	4000	11	-	28
7 – 8		3000	9	-	28
8 – 9		5100	15	-	30
9 – 10		12000	34	-	34
10 – 11	Clay to silty clay	3000	17	80	-
11 – 12		3200	18	85	-
12 – 13		4000	23	105	-
13 – 14		4000	23	105	-
14 - 15		3800	22	100	-
15 – 16.2		3800	22	115	-

ECPT – T6					
Depth (m)	**Soil Strata**	**Cone resistance (qc) [kPa]**	**N value**	**Cohesion (kPa)**	**Φ (degree)**
0 – 1	Sand mixtures	2000	9	-	28
1 – 2		1800	8	-	28
2 – 3		1100	5	-	26
3 – 4		1000	4	-	26
4 – 5		1000	4	-	26
5 - 6	Sensitive fine grained soil	600	3	20	-
6 – 7	Sands	6000	17	-	30
7 – 8		9000	26	-	32
8 – 9		7000	20	-	32
9 – 10	Clay to silty clay	2000	11	65	-
10 – 11	sands	4600	13	-	30
11 – 12		10000	29	-	34
12 – 13		7500	21	-	30
13 – 14	Clay to silty clay	4000	23	105	-
14 - 15		3600	21	100	-
15 – 15.6		3600	21	100	-

ECPT – T7					
Depth (m)	**Soil Strata**	**Cone resistance (qc) [kPa]**	**N value**	**Cohesion (kPa)**	**Φ (degree)**
0 – 1		-	-	-	-
1 – 2	Organic soil – clay	0	0	0	-
2 – 3	Clay to silty clay	500	3	15	-
3 – 4	Sands	2500	7	-	28
4 – 5	Clay to silty clay	1900	11	50	-
5 - 6		2100	12	75	-
6 – 7		2000	11	70	-
7 – 8		2500	14	90	-
8 – 9		3100	18	95	-
9 – 10		3000	17	90	-
10 – 11		3600	21	110	-
11 – 12.5		7000	40	250	-

ECPT – T8					
Depth (m)	**Soil Strata**	**Cone resistance (qc) [kPa]**	**N value**	**Cohesion (kPa)**	**Φ (degree)**
0 – 1	Stiff sand to clayey sand	3000	13	-	30
1 – 2	Sand mixtures	1300	6	-	26
2 – 3	Clayey silt to silty clay	1100	6	30	-
3 – 4	Clay to silty clay	1000	6	35	-
4 – 5		1000	6	35	-
5 - 6		1000	6	30	-
6 – 7		2000	11	50	-
7 – 8		3100	18	85	-
8 – 9	Clayey silt to silty clay	5100	29	130	-
9 – 10		3500	20	105	-
10 – 11	Clay to silty clay	3100	18	95	-
11 – 12		3500	20	90	-
12 – 13		7500	43	200	-
13 – 14		3100	18	85	-
14 - 15		3500	20	93	-
15 – 16.1		3900	22	100	-

ECPT – T9					
Depth (m)	**Soil Strata**	**Cone resistance (qc) [kPa]**	**N value**	**Cohesion (kPa)**	**Φ (degree)**
0 – 1	Clay to silty clay	100	1	6.4	-
1 – 2		600	3	20	-
2 – 3		500	3	20	-
3 – 4		200	1	6.4	-
4 – 5		1000	6	40	-
5 - 6		200	1	6.4	-
6 – 7		1000	6	40	-
7 – 8	sands	9000	26	-	32
8 – 9		5000	14	-	28
9 – 10	Clay to silty clay	1100	6	25	-
10 – 11		2000	11	50	-
11 – 12		2500	14	65	-
12 – 13		2600	15	75	-
13 – 14		2500	14	70	-
14 - 15		2500	14	70	-
15 – 16.2		2600	15	70	-

ECPT – T10					
Depth (m)	**Soil Strata**	**Cone resistance (qc) [kPa]**	**N value**	**Cohesion (kPa)**	**Φ (degree)**
0 – 1	Sand mixtures	3000	13	-	30
1 – 2		1000	4	-	26
2 – 3	Fine grained soil	500	3	20	-
3 – 4		800	5	32	-
4 – 5		1000	6	40	-
5 - 6		1000	6	40	-
6 – 7	Sands	8000	23	-	32
7 – 8		12000	34	-	36
8 – 9	Clay to silty clay	3500	20	105	-
9 – 10		3200	18	95	-
10 – 11		3500	20	105	-
11 – 12		3000	17	90	-
12 – 13.5		4500	26	135	-

ECPT – T11					
Depth (m)	**Soil Strata**	**Cone resistance (qc) [kPa]**	**N value**	**Cohesion (kPa)**	**Φ (degree)**
0 – 1	Gravelly sand to sand	6500	19	-	30
1 – 2	Very stiff fine grained	3000	17	90	-
2 – 3	Clayey silt to silty clay	4000	23	120	-
3 – 4	Clay to silty clay	2000	11	70	-
4 – 5	Organic soil – peat	1500	9	40	-
5 - 6		1000	6	30	-
6 – 7	Clay to silty clay	1600	9	60	-
7 – 8		3000	17	90	-
8 – 9		3100	18	95	-
9 – 10		3500	20	105	-
10 – 11.1		3800	22	115	-

ECPT – T12					
Depth (m)	**Soil Strata**	**Cone resistance (qc) [kPa]**	**N value**	**Cohesion (kPa)**	**Φ (degree)**
0 – 1	Sands	2000	6	-	26
1 – 2	Sand mixtures	1000	4	-	26
2 – 3		1000	4	-	26
3 – 4		1100	5	-	26
4 – 5		1000	4	-	26
5 - 6		700	3	-	26
6 – 7	Sands	5100	15	-	30
7 – 8		8000	23	-	32
8 – 9		12000	34	-	34
9 – 10		13500	39	-	34
10 – 11	Clay to silty clay	4000	23	110	-
11 – 12		3000	17	80	-
12 – 13		3500	20	95	-
13 – 14		3700	21	100	-
14 - 15		3800	22	110	-
15 – 16		4000	23	125	-
16 – 16.7		4100	23	125	-

ECPT – T13					
Depth (m)	**Soil Strata**	**Cone resistance (qc) [kPa]**	**N value**	**Cohesion (kPa)**	**Φ (degree)**
0 – 1	Sand mixtures	2300	10	-	28
1 – 2		1500	7	-	28
2 – 3		1000	4	-	26
3 – 4	Fine grained soil	1100	6	40	-
4 – 5		1000	6	40	-
5 - 6		500	3	20	-
6 – 7		700	4	30	-
7 – 8		600	3	20	-
8 – 9	Clay to silty clay	2000	11	55	-
9 – 10		2100	12	60	-
10 – 11		2200	13	60	-
11 – 12		3000	17	80	-
12 – 13		3000	17	80	-
13 – 14		2900	17	83	-
14 - 15		2500	14	70	-
15 – 15.9		3000	17	91	-

ENCLOSURE 3

COMPARISON OF SOIL STRATA FROM NEARBY ECPTs AND SPTs

<table>
<tr><td colspan="5">Borehole – C7, Nearest ECPT – T9
Distance between C7 and T9 = 110 m</td></tr>
<tr><td rowspan="2">Depth (m)</td><td colspan="2">BH – C7</td><td colspan="2">ECPT – T9</td></tr>
<tr><td>Ground elevation – 4.73 m
Water table – 0.7 m</td><td>SPT N</td><td>Ground elevation – 4.65 m
Water table – 1.2 m</td><td>Interpreted SPT N</td></tr>
<tr><td>0 – 1</td><td rowspan="6">Very soft clay</td><td rowspan="6">0</td><td>Not defined</td><td>1</td></tr>
<tr><td>1 – 1.2</td><td>Clayey silt to silty clay</td><td>3</td></tr>
<tr><td>1.2 – 1.4</td><td>Very stiff fine-grained soil</td><td>3</td></tr>
<tr><td>1.4 – 2.65</td><td>Clay to silty clay</td><td>3</td></tr>
<tr><td>2.65 – 3</td><td>Organic peat</td><td>3</td></tr>
<tr><td>3 – 4.5</td><td rowspan="2">Clay to silty clay</td><td>1</td></tr>
<tr><td>4.5 – 6.65</td><td rowspan="3">Silty sand</td><td rowspan="3">16</td><td>6</td></tr>
<tr><td>6.65 - 9</td><td>sands</td><td>20</td></tr>
<tr><td>9 – 10.5</td><td rowspan="3">Clay</td><td rowspan="2">11</td></tr>
<tr><td>10.5 – 13.5</td><td>Stiff silty clay</td><td>9</td></tr>
<tr><td>13.5 – 16.5</td><td rowspan="2">Very Stiff clay</td><td rowspan="2">19</td><td>14</td></tr>
<tr><td>16.5 – 18</td><td colspan="2" rowspan="4"></td></tr>
<tr><td>18 – 19.5</td><td>Silty sand</td><td>20</td></tr>
<tr><td>19.5 – 28.5</td><td>Hard clay</td><td>28</td></tr>
<tr><td>28.5 - 40</td><td>Dense sand</td><td>>50</td></tr>
</table>

<table>
<tr><td colspan="5">Borehole – C1, Nearest ECPT – T1
Distance between C1 and T1 = 110 m</td></tr>
<tr><th rowspan="2">Depth (m)</th><th colspan="2">BH – C1</th><th colspan="2">ECPT – T1</th></tr>
<tr><td>Ground elevation – 4.99 m
Water table – 0.6 m</td><th>SPT N</th><td>Ground elevation – 4.78 m
Water table – 1.2 m</td><th>Interpreted SPT N</th></tr>
<tr><td>0 – 0.5</td><td rowspan="3">Very soft clay</td><td rowspan="3">1, 2</td><td>Very stiff fine-grained soil</td><td>3</td></tr>
<tr><td>0.5 – 3</td><td>Clayey silt to silty clay</td><td>3</td></tr>
<tr><td>3 – 4.5</td><td>sands</td><td>5</td></tr>
<tr><td>4.5 – 5.3</td><td rowspan="2">Sandy silt</td><td rowspan="2">2</td><td>Clayey silt to silty clay</td><td>6</td></tr>
<tr><td>5.3 – 7.5</td><td>sands</td><td>23</td></tr>
<tr><td>7.5 - 9</td><td>Fine sand</td><td>22</td><td rowspan="4">Clay to silty clay</td><td>14</td></tr>
<tr><td>9 – 10.5</td><td>Stiff silty clay</td><td>22</td><td>17</td></tr>
<tr><td>10.5 – 12</td><td>Hard silty clay</td><td>32</td><td>18</td></tr>
<tr><td>12 – 15</td><td rowspan="3">Very Stiff silty clay</td><td>18</td><td>20</td></tr>
<tr><td>15 – 16.5</td><td>25</td><td colspan="2" rowspan="3"></td></tr>
<tr><td>16.5 – 27</td><td>23</td></tr>
<tr><td>27 - 40</td><td>Dense sand</td><td>>50</td></tr>
</table>

<table>
<tr><td colspan="5">Borehole – C3, Nearest ECPT – T2
Distance between C3 and T2 = 90 m</td></tr>
<tr><td rowspan="2">Depth (m)</td><td colspan="2">BH – C3</td><td colspan="2">ECPT – T2</td></tr>
<tr><td>Ground elevation – 6.4 m
Water table – 1.2 m</td><td>SPT N</td><td>Ground elevation – 6.44 m
Water table – 1.5 m</td><td>Interpreted SPT N</td></tr>
<tr><td>0 – 1</td><td>Salt pan bund fill</td><td>7</td><td>Very stiff fine grained</td><td>6</td></tr>
<tr><td>1 – 2.5</td><td rowspan="3">Very soft clay</td><td rowspan="3">0</td><td>Clay to silty clay</td><td>9</td></tr>
<tr><td>2.5 - 3</td><td>Organic soil peat</td><td>3</td></tr>
<tr><td>3 – 7.5</td><td rowspan="3">Clay to silty clay</td><td>4</td></tr>
<tr><td>7.5 – 10.5</td><td>Soft clay</td><td>1</td><td>13</td></tr>
<tr><td>10.5 – 11.4</td><td rowspan="2">Silty clay</td><td rowspan="2">14</td><td>14</td></tr>
<tr><td>11.4 - 12</td><td colspan="2" rowspan="9"></td></tr>
<tr><td>12 – 22.5</td><td>stiff silty clay</td><td>25</td></tr>
<tr><td>22.5 - 24</td><td>Clayey silt</td><td>33</td></tr>
<tr><td>24 – 25.5</td><td>Clayey silty sand</td><td>38</td></tr>
<tr><td>25.5 – 28.5</td><td>Silty clay</td><td>30</td></tr>
<tr><td rowspan="4">28.5 - 40</td><td rowspan="4">Dense silty sand</td><td>36</td></tr>
<tr><td>44</td></tr>
<tr><td>51</td></tr>
<tr><td>>50</td></tr>
</table>

ENCLOSURE 1

Safe Bearing Capacity Calculations

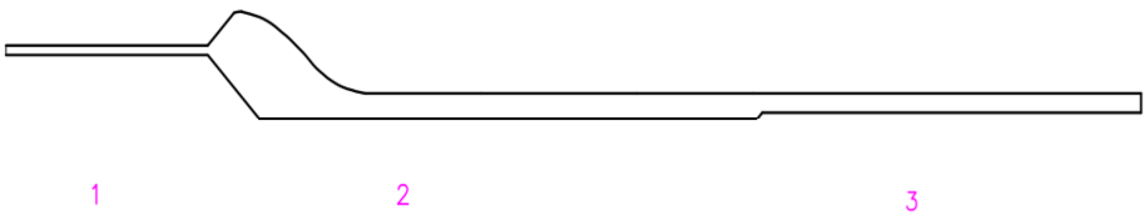


Figure A2.1: Different sections considered for SBC calculation

a) Safe Bearing capacity for section 1

Bore Hole	=		BH C7
Ground water table location (Dw)	=	0.0	m
Unit weight of soil (weighted average) (ϒ)	=	1.8	t/m^3
Submerged unit weight of soil (g')	=	1	t/m^3
Average SPT $N_{corrected}$ value (Considered upto 1.5 B (as per IS 6403 – 1981))	=		27
Type of Foundation	=		Raft Foundation
Founding Level	=	11	m from RL of Borehole
Type of Analysis N corrected ≥ 30		**General Shear Failure**	
Thickness of Raft (Df)	=	1	m
Length of Raft (L)	=	26	m
Width of Raft (B)	=	18.0	m
Angle of internal friction (ø) at founding level	=	35	deg.
Cohesion I	=	6.3	t/m^2
Bearing capacity factors (Weighted average)			
Nc	=		19.56
Nq	=		12.36
$N_ϒ$	=		16.90
Shape factors			
Sc	=		1.14 (From Table 2 IS:6403)
Sq	=		1.14
$S_ϒ$	=		0.72
Depth factors			
dc	=		1.02
dq	=		1.01
$d_ϒ$	=		1.01
Effective overburden pressure (q)	=		1 t/m^2
Factor of safety (FoS)	=		2.5

Ultimate Bearing Capacity (UBC)

UBC = C *Nc*Sc*dc*ic + q * (Nq – 1)* Sq * dq * iq +0.5 *B*ϒ * $N_ϒ$ * $S_ϒ$* $d_ϒ$ * $i_ϒ$ *w

Where, w	=	Water table correction factor
w	=	0.50
Ultimate Bearing Capacity	=	256 t/m^2

Safe bearing capacity as per General Shear Failure criteria,

(SBC) = UBC/FoS = **103 t/m^2**

Bore Hole	=		BH C7
Ground water table location (Dw)	=	0.0	m
Unit weight of soil (weighted average) (ϒ)	=	1.8	t/m^3
Submerged unit weight of soil **(γ')**	=	1	t/m^3
Average SPT Ncorrected value	=		27
(Considered upto 1.5 B (as per IS 6403 – 1981))			
Type of Foundation:	=		Raft Foundation
Founding Level	=	11	m
Type of Analysis N corrected ≤ 10		**Local Shear Failure**	
Thickness of Raft (Df)	=	1	m
Length of Raft (L)	=	26	m
Width of Raft (B)	=	18	m
Angle of internal friction (φ)	=	35	deg.
Angle of internal friction for LSF (φ' = tan-1(2/3 tan φ)	=	25	deg.
Cohesion I	=	6.3	t/m^2
Bearing capacity factors			
Nc	=		10.63
Nq	=		4.41
Nϒ	=		3.85
Shape factors			
Sc	=		1.14 (From Table 2 IS:6403)
Sq	=		1.14
Sϒ	=		0.72
Depth factors			
dc	=		1.017
dq	=		1.009
dϒ	=		1.009
Effective overburden pressure (q)	=		1 t/m^2
Factor of safety (FoS)	=		2.5

Ultimate Bearing Capacity (UBC)

UBC = 2/3*C *Nc*Sc*dc*ic + q * (Nq – 1)* Sq * dq * iq +0.5 *B*ϒ * $N_ϒ$ * $S_ϒ$* $d_ϒ$* $i_ϒ$ *w

Where, w	=	Water table correction factor
w	=	0.50
Ultimate Bearing Capacity	=	78.3 t/m^2

Safe bearing capacity as per Local Shear Failure criteria,
(SBC) = UBC/FoS = **31.3 t/m^2**

Note:

The SBC is calculated by general shear failure method (GSF) for N > 30 & by local shear failure method (LSF) for N < 10.

For the present case, N avg (corrected) considered upto 1.5 times width of the footing is 27. The SBC calculated by GSF method is 103 t/m^2 and by LSF method is 31 t/m^2. Linearly interpolating for N avg (corrected) value of 27, SBC of raft works out to 90 t/m^2.

Table A2.1: Bearing Capacity for Section-1

Bearing capacity calculation based on shear criteria for Section 1			
	L.S.F ($N \leq 10$)	G.S.F ($N \geq 30$)	**$10 \leq N \leq 30$**
Navg (corrected)	10	30	**27**
SBC (t/m²)	31	103	**90**

b) Safe Bearing capacity for section 2

Bore Hole	=	BH C7	
Ground water table location (D_w)	=	0.0	m
Unit weight of soil (weighted average) (ϒ)	=	1.8	t/m³
Submerged unit weight of soil (g')	=	1	t/m³
Average SPT N corrected value	=	29	
(Considered upto 1.5 B (as per IS 6403 - 1981))			
Type of Foundation	=	Raft Foundation	
Founding Level	=	17.5	m from RL of Borehole
Type of Analysis N corrected ≥ 30		General Shear Failure	
Thickness of Raft (Df)	=	2.5	m
Length of Raft (L)	=	1.0	m
Width of Raft (B)	=	67.0	m
Angle of internal friction (ø) at founding level	=	35	deg.
Cohesion (C)	=	1.27	t/m²
Bearing capacity factors (Weighted average)			
Nc	=	41.43	
Nq	=	29.60	
N_Υ	=	42.53	
Shape factors			
Sc	=	1.00	(From Table 2 IS:6403)
Sq	=	1.00	
S_Υ	=	1.00	
Depth factors			
dc	=	1.01	
dq	=	1.01	
d_Υ	=	1.01	
Effective overburden pressure (q)	=	2.5 t/m²	
Factor of safety (FoS)	=	2.5	

Ultimate Bearing Capacity (UBC)

$UBC = C * Nc*Sc*dc*ic + q * (Nq - 1)* Sq * dq * iq + 0.5 * B*\Upsilon * N_\Upsilon * S_\Upsilon * d_\Upsilon * i_\Upsilon * w$

where, w	=	Water table correction factor	
w	=	0.50	
Ultimate Bearing Capacity	=	1417	t/m²

Safe bearing capacity as per General Shear Failure criteria,

(SBC) = UBC/FoS = **567** **t/m²**

Bore Hole	=	BH C7	
Ground water table location (D_W)	=	0.0	m
Unit weight of soil (weighted average) (Υ)	=	1.8	t/m^3
Submerged unit weight of soil (g')	=	1	t/m^3
Average SPT N corrected value	=	29	
(Considered upto 1.5 B (as per IS 6403 - 1981))			
Type of Foundation	=	Raft Foundation	
Founding Level	=	17.5	m from RL of Borehole
Type of Analysis N corrected ≥ 30		Local Shear Failure	
Thickness of Raft (Df)	=	2.5	m
Length of Raft (L)	=	1.0	m
Width of Raft (B)	=	67.0	m
Angle of internal friction (ø)	=	35	deg.
Angle of internal friction for LSF ($\varphi' = \tan^{-1}(2/3 \tan \varphi)$)		25	Deg.
Cohesion (C)	=	1.27	t/m^2
Bearing capacity factors (Weighted average)			
Nc	=	18.33	
Nq	=	9.17	
N_Υ	=	9.21	
Shape factors			
Sc	=	1.00	(From Table 2 IS:6403)
Sq	=	1.00	
S_Υ	=	1.00	
Depth factors			
dc	=	1.012	
dq	=	1.006	
d_Υ	=	1.006	
Effective overburden pressure (q)	=	2.5 t/m^2	
Factor of safety (FoS)	=	2.5	

Ultimate Bearing Capacity (UBC)

$UBC = C * Nc * Sc * dc * ic + q * (Nq - 1) * Sq * dq * iq + 0.5 * B * \Upsilon * N_\Upsilon * S_\Upsilon * d_\Upsilon * i_\Upsilon * w$

where, w	=	Water table correction factor	
w	=	0.50	
Ultimate Bearing Capacity	=	315.6	t/m^2

Safe bearing capacity as per General Shear Failure criteria,

(SBC) = UBC/FoS = **126.2** **t/m^2**

Table A2.2: Bearing Capacity for Section-2

Bearing capacity calculation based on shear criteria for Section 1			
	L.S.F (N ≤ 10)	G.S.F (N ≥ 30)	**10 ≤ N ≤ 30**
Navg (corrected)	10	30	**27**
SBC (t/m^2)	126	567	**550**

c) Safe Bearing capacity for section 3

Bore Hole	=	BH C7	
Ground water table location (D_w)	=	0.0	m
Unit weight of soil (weighted average) (Υ)	=	1.8	t/m^3
Submerged unit weight of soil (g')	=	1	t/m^3
Average SPT N corrected value	=	29	
(Considered upto 1.5 B (as per IS 6403 - 1981))			
Type of Foundation	=	Raft Foundation	
Founding Level	=	17.0	m from RL of Borehole
Type of Analysis N corrected ≥ 30		General Shear Failure	
Thickness of Raft (Df)	=	2.0	m
Length of Raft (L)	=	1.0	m
Width of Raft (B)	=	50.0	m
Angle of internal friction (ø) at founding level	=	35	deg.
Cohesion (C)	=	1.7	t/m^2
Bearing capacity factors (Weighted average**)**			
Nc	=	39.84	
Nq	=	28.35	
N_Υ	=	40.67	
Shape factors			
Sc	=	1.00	(From Table 2 IS:6403)
Sq	=	1.00	
S_Υ	=	1.00	
Depth factors			
dc	=	1.02	
dq	=	1.01	
d_Υ	=	1.01	
Effective overburden pressure (q)	=	2.5 t/m^2	
Factor of safety (FoS)	=	2.5	

Ultimate Bearing Capacity (UBC)

$$UBC = C * Nc*Sc*dc*ic + q * (Nq - 1)* Sq * dq * iq + 0.5 *B*\Upsilon * N_\Upsilon * S_\Upsilon * d_\Upsilon * i_\Upsilon *w$$

where, w	=	Water table correction factor	
w	=	0.50	
Ultimate Bearing Capacity	=	1046	t/m^2
Safe bearing capacity as per General Shear Failure criteria, (SBC) = UBC/FoS	=	**418**	**t/m^2**

Bore Hole	=	BH C7	
Ground water table location (D_w)	=	0.0	m
Unit weight of soil (weighted average) (Υ)	=	1.8	t/m^3
Submerged unit weight of soil (g')	=	1	t/m^3
Average SPT N corrected value	=	29	
(Considered upto 1.5 B (as per IS 6403 - 1981))			
Type of Foundation	=	Raft Foundation	

Founding Level	=	17.0	m from RL of Borehole
Type of Analysis N corrected ≥ 30		Local Shear Failure	
Thickness of Raft (Df)	=	2	m
Length of Raft (L)	=	1.0	m
Width of Raft (B)	=	50.0	m
Angle of internal friction (ø)	=	35	deg.
Angle of internal friction for LSF ($\varphi' = \tan^{-1}(2/3 \tan \varphi)$)		25	Deg.
Cohesion (C)	=	1.7	t/m^2
Bearing capacity factors (Weighted average)			
N_c	=	18.33	
N_q	=	9.17	
N_γ	=	9.21	
Shape factors			
S_c	=	1.00	(From Table 2 IS:6403)
S_q	=	1.00	
S_γ	=	1.00	
Depth factors			
d_c	=	1.012	
d_q	=	1.006	
d_γ	=	1.006	
Effective overburden pressure (q)	=	2.5 t/m^2	
Factor of safety (FoS)	=	2.5	

Ultimate Bearing Capacity (UBC)

$$UBC = C * N_c * S_c * d_c * i_c + q * (N_q - 1) * S_q * d_q * i_q + 0.5 * B * \gamma * N_\gamma * S_\gamma * d_\gamma * i_\gamma * w$$

where, w	=	Water table correction factor	
w	=	0.50	
Ultimate Bearing Capacity	=	246	t/m^2
Safe bearing capacity as per General Shear Failure criteria, (SBC) = UBC/FoS	=	**98**	**t/m^2**

Table A2.3: Bearing Capacity for Section-3

Bearing capacity calculation based on shear criteria for Section 1			
	L.S.F (N ≤ 10)	G.S.F (N ≥ 30)	**10 ≤ N ≤ 30**
Navg (corrected)	10	30	**27**
SBC (t/m^2)	98	418	**405**

d) Bearing Capacity for upstream Flank wall vertical section (S1):

Ground water table location (D_w)	=	0.0 m
Unit weight of soil (weighted average) (Υ)	=	1.8 t/m^3
Submerged unit weight of soil (g')	=	1 t/m^3
Average SPT $N_{corrected}$ value	=	-
(Considered upto 1.5 B (as per IS 6403 - 1981))		
Type of Foundation	=	Raft Foundation
Founding Level	=	10 m
Type of Analysis N corrected ≥ 30 General Shear Failure		
Thickness of Raft (D_f)	=	2 m
Length of Raft (L)	=	1.0 m
Width of Raft (B)	=	37 m
Angle of internal friction (ø) at founding level	=	5°
Cohesion (C)	=	17.6 t/m^2
Bearing capacity factors (Weighted average)		
N_c	=	6.49
N_q	=	1.57
N_Υ	=	0.45
Shape factors (From Table - 2 IS: 6403)		
S_c	=	1.00
S_q	=	1.00
S_Υ	=	1.00
Depth factors		
d_c	=	1.00
d_q	=	1.00
$d\Upsilon$	=	1.00
Effective overburden pressure (q)	=	2 t/m^2
Factor of safety (FoS)	=	2.5

Ultimate Bearing Capacity (UBC)

$UBC = C * N_c * S_c * d_c * i_c + q * (N_q - 1) * S_q * d_q * i_q + 0.5 * B * \Upsilon * N_\Upsilon * S_\Upsilon * d_\Upsilon * i_\Upsilon * w$

Where,

w = Water table correction factor

w = 0.50

Ultimate Bearing Capacity = 124 t/m^2

Safe bearing capacity as per General Shear Failure criteria,

(SBC) = $\frac{UBC}{FoS}$ = **50 t/m^2**

e) Bearing Capacity for downstream Flank wall vertical section (S1):

Ground water table location (D_w)	=	0.0 m
Unit weight of soil (weighted average) (Υ)	=	1.8 t/m^3
Submerged unit weight of soil (g')	=	1 t/m^3
Average SPT $N_{corrected}$ value	=	33
(Considered upto 1.5 B (as per IS 6403 - 1981))		
Type of Foundation	=	Raft Foundation
Founding Level	=	10
Type of Analysis N corrected ≥ 30 General Shear Failure		
Thickness of Raft (D_f)	=	1.3 m
Length of Raft (L)	=	1.0 m
Width of Raft (B)	=	55.1 m
Angle of internal friction (ø) at founding level	=	32°
Cohesion (C)	=	2.28 t/m^2
Bearing capacity factors (Weighted average)		
N_c	=	29.11
N_q	=	18.84
N_Υ	=	24.93
Shape factors (From Table - 2 IS: 6403)		
S_c	=	1.00
S_q	=	1.00
S_Υ	=	1.00
Depth factors		
d_c	=	1.00
d_q	=	1.00
d_Υ	=	1.00
Effective overburden pressure (q)	=	1.3 t/m^2
Factor of safety (FoS)	=	2.5

Ultimate Bearing Capacity (UBC)

$UBC = C * N_c * S_c * d_c * i_c + q * (N_q - 1) * S_q * d_q * i_q + 0.5 * B * \Upsilon * N_\Upsilon * S_\Upsilon * d_\Upsilon * i_\Upsilon * w$

Where,

w	=	Water table correction factor
w	=	0.50
Ultimate Bearing Capacity	=	707 t/m^2

Safe bearing capacity as per General Shear Failure criteria,

(SBC) = $\frac{UBC}{FoS}$ = **283 t/m^2**

ENCLOSURE 2

Calculations of Pile Capacities, Number and Spacing of piles

(i) Pile capacity calculations for 1 m dia., 20 m length of pile

Type of pile:	Bored Cast in-situ concrete piles	
Diameter of pile	1	m
Cut-off Level of pile	17.5	m below Existing Ground Level
Length of the pile, below cut-off level	20	m
Termination level of the pile	37.5	m
Depth of GWT, below ground level	0	m
Depth of GWT, below cut off level	0	m
Bore Hole No	BH C1	
Bulk Unit weight of soil	18	kN/m^3
Unit weight of water	10	kN/m^3
Submerged Weight of soil	10	kN/m^3
Angle of friction between pile & soil, δ	Φ	
Bearing capacity factor, N_γ	48.03	(from IS:6403, table -1) Φ = 35 deg for End bearing
Bearing capacity factor, N_q	48	(from IS:2911-part-1-sec-2-Fig.1)
Area of pile at tip, A_p	0.785	m^2
Adhesion Factor (α)	0.45	(from IS:2911-part-1-sec-2-Appendix B 2.1, Note 1)
Bearing Capacity Factor, N_c	9	

Soil strata:

Layer	Depth (m)	Thickness (m)	Soil type	Φ (°)	Cohesion (kPa)	Adhesion factor (α)	Earth pressure coefficient
Layer I	17.5 - 27	9.5	Stiff silty clay	0	101.0	0.45	1
Layer II	27 – 37.5	10.5	Sand	35	0	1	1

Pile Capacity, Qu = Ap [(0.5 x D x γ x Nγ) + (Pd x Nq)+(Nc x Cp)] +
[(ΣD i=1 K x Pdi x tan δ x Asi)+(αC'x As)]

Critical depth, D_c = 15 x dia of pile 15 x 1 m = 15 m

Effective Overburden Pressure at Critical Depth = 150 kN/m^2

Effective Overburden Pressure at tip of the pile = 200

Pressure calculations:

Depth below ground level / Cut-off level	Depth (m)	Linear Pressure variation (kN/m^2)	Pressure variation after considering critical depth (kN/m^2)
17.5 m / 0 m	0	0	0
27 m / 9.5 m	9.5	95	95
32.5 m / 15 m	20	200	150

Calculation of Friction component:

Layer- I

Qs1 = $[(\Sigma D\ i=1\ K \times P_{di} \times \tan\delta \times A_{si})+(\alpha C' \times As)]$

Qs1 = 1 x [(0 + 95)/2 x tan 0 x π x 1 x 9.5] + (0.45 x 101 x π x 1 x 9.5)

Qs1 = 1356.5 kN

Layer- I

Qs1 = $[(\Sigma D\ i=1\ K \times P_{di} \times \tan\delta \times A_{si})+(\alpha\ C' \times As)]$

Qs1 = 1 x [(95 + 200)/2 x tan 35 x π x 1 x 10.5] + (1 x 0 x π x 1 x 10.5)

Qs1 = 3407 kN

Skin friction due to soil = Qs = sum (Qs1 to Qs2) = 4763 kN

Safe skin friction due to soil = $\frac{Qs}{FoS}$ = $\frac{4763}{2.5}$ = 191 tons

Calculation of End bearing Component:

Qe = Ap [(0.5 x D x ɣ x Nɣ) + (Pd x Nq) + (Nc x Cp)]

Qe = 0.785 [(0.5 x 1 x 10 x 48.03) + (150 x 48) + (9 x 0)] = 5804.5 kN

Safe end bearing component of pile = $\frac{Qe}{FoS}$ = $\frac{5804.5}{3}$ = 234 tons

Safe Axial capacity of pile = Qs + Qe = 191 t + 234 t = **425 tons**

Uplift Pile Capacities:

Submerged Unit weight of Concrete (Υsub)		=	15 kN/m^3	
Weight of pile (W)		=	235.5 kN	
Total Skin friction = Qs		=	4763.4 kN	
Ultimate uplift capacity of pile, Qu =	Qs + W		=	4763.5/3 + 235.5 kN
Ultimate uplift capacity of pile, Qu			**=**	**182 tons**

(ii) Pile capacity calculations for 1.5 m dia., 20 m length of pile

Type of pile:	Bored Cast in-situ concrete piles	
Diameter of pile	1.5	m
Cut-off Level of pile	17.5	m below Existing Ground Level
Length of the pile, below cut-off level	20	m
Termination level of the pile	37.5	m
Depth of GWT, below ground level	0	m
Depth of GWT, below cut off level	0	m
Bore Hole No	BH C1	
Bulk Unit weight of soil	18	kN/m^3
Unit weight of water	10	kN/m^3
Submerged Weight of soil	10	kN/m^3
Angle of friction between pile & soil, δ	Φ	
Bearing capacity factor, N_γ	48.03	(from IS:6403, table -1) Φ = 35 deg for End bearing
Bearing capacity factor, N_q	48	(from IS:2911-part-1-sec-2-Fig.1)
Area of pile at tip, A_p	1.766	m^2
Adhesion Factor (α)	0.45	(from IS:2911-part-1-sec-2-Appendix B 2.1, Note 1)
Bearing Capacity Factor, N_c	9	

Soil strata:

Layer	Depth (m)	Thickness (m)	Soil type	Φ (°)	Cohesion (kPa)	Adhesion factor (α)	Earth pressure coefficient
Layer I	17.5 - 27	9.5	Stiff silty clay	0	101.0	0.45	1
Layer II	27 – 37.5	10.5	Sand	35	0	1	1

Pile Capacity, Qu = Ap [(0.5 x D x ƴ x Nƴ) + (Pd x Nq)+(Nc x Cp)] + [(ΣD i=1 K x Pdi x tan δ x Asi)+(αC'x As)]

Critical depth, Dc = 15 x dia of pile 15 x 1.5 m = 22.5 m

Effective Overburden Pressure at Critical Depth = 225 kN/m²

Effective Overburden Pressure at tip of the pile = 200

Pressure calculations:

Depth below ground level / Cut-off level	Depth (m)	Linear Pressure variation (kN/m²)	Pressure variation after considering critical depth (kN/m²)
17.5 m / 0 m	0	0	0
27 m / 9.5 m	9.5	95	95
32.5 m / 15 m	20	200	200

Calculation of Friction component:

Layer- I

Qs1 = [(ΣD i=1 K x P_{di} x tan δ x A_{si})+(α C' x As)]

Qs1 = 1 x [(0 + 95)/2 x tan 0 x π x 1.5 x 9.5] + (0.45 x 101 x π x 1.5 x 9.5)

Qs1 = 2034.7 kN

Layer- I

Qs1 = [(ΣD i=1 K x P_{di} x tan δ x A_{si})+(α C' x As)]

Qs1 = 1 x [(95 + 200)/2 x tan 35 x π x 1.5 x 10.5] + (1 x 0 x π x 1.5 x 10.5)

Qs1 = 5110.3 kN

Skin friction due to soil = Qs = sum (Qs1 to Qs2) = 7145 kN

Safe skin friction due to soil = $\frac{Qs}{FoS}$ = $\frac{7145}{2.5}$ = 286 tons

Calculation of End bearing Component:

Qe = Ap [(0.5 x D x γ x Nγ) + (Pd x Nq) + (Nc x Cp)]

Qe = 1.766 [(0.5 x 1.5 x 10 x 48.03) + (200 x 48) + (9 x 0)] = 17589.8 kN

Safe end bearing component of pile = $\frac{Qe}{\boldsymbol{FoS}} = \frac{17589.8}{3}$ = 704 tons

Safe Axial capacity of pile = Qs + Qe = 286 t + 704 t = **990 tons**

Uplift Pile Capacities:

Submerged Unit weight of Concrete (Ysub) = 15 kN/m^3

Weight of pile (W) = 529.8 kN

Total Skin friction = Qs = 7145 kN

Ultimate uplift capacity of pile, Qu = Qs + W = 7145/3 + 529.8 kN

Ultimate uplift capacity of pile, Qu = 291 tons

Calculation of number of piles and spacing:

Upward pressure due to uplift = 10.3 t/m^2

Area of the raft = 67 x 2200 = 147400 m^2

Total uplift = 10.3 x 147400 = 1.52 x 106 t

i. *<u>For C1 Borehole:</u>*

Taking diameter of pile = 1 m

Length of pile = 20 m

Uplift capacity = 182 t

Number of piles = $\frac{Total\ uplift}{Uplift\ capacity}$

Number of piles = $\frac{1.52\ x\ 10^6}{182}$ = 8342

Spacing = $\sqrt{\frac{Area\ of\ the\ raft}{Number\ of\ piles}}$

Spacing = $\sqrt{\frac{147400}{8342}}$ = 4.2 m

ii. *<u>For C2 Borehole:</u>*

Taking diameter of pile = 1 m

Length of pile = 20 m

Uplift capacity = 138 t

Number of piles = $\frac{Total\ uplift}{Uplift\ capacity}$

Number of piles = $\frac{1.52\ x\ 10^6}{138}$ = 11015

Spacing = $\sqrt{\frac{Area\ of\ the\ raft}{Number\ of\ piles}}$

Spacing = $\sqrt{\frac{147400}{11015}}$ = 3.7 m

iii. *For C3 Borehole:*

Taking diameter of pile = 1 m

Length of pile = 20 m

Uplift capacity = 160 t

Number of piles = $\frac{Total\ uplift}{Uplift\ capacity}$

Number of piles = $\frac{1.52\ x\ 10^6}{160}$ = 9500

Spacing = $\sqrt{\frac{Area\ of\ the\ raft}{Number\ of\ piles}}$

Spacing = $\sqrt{\frac{147400}{9500}}$ = 3.94 m

iv. *For C7 Borehole:*

Taking diameter of pile = 1 m

Length of pile = 20 m

Uplift capacity = 186 t

Number of piles = $\frac{Total\ uplift}{Uplift\ capacity}$

Number of piles = $\frac{1.52\ x\ 10^6}{186}$ = 8172

Spacing = $\sqrt{\frac{Area\ of\ the\ raft}{Number\ of\ piles}}$

Spacing = $\sqrt{\frac{147400}{8172}}$ = 4.25 m

ENCLOSURE 3

Calculations of Sheet pile Cofferdam

Borehole – D4

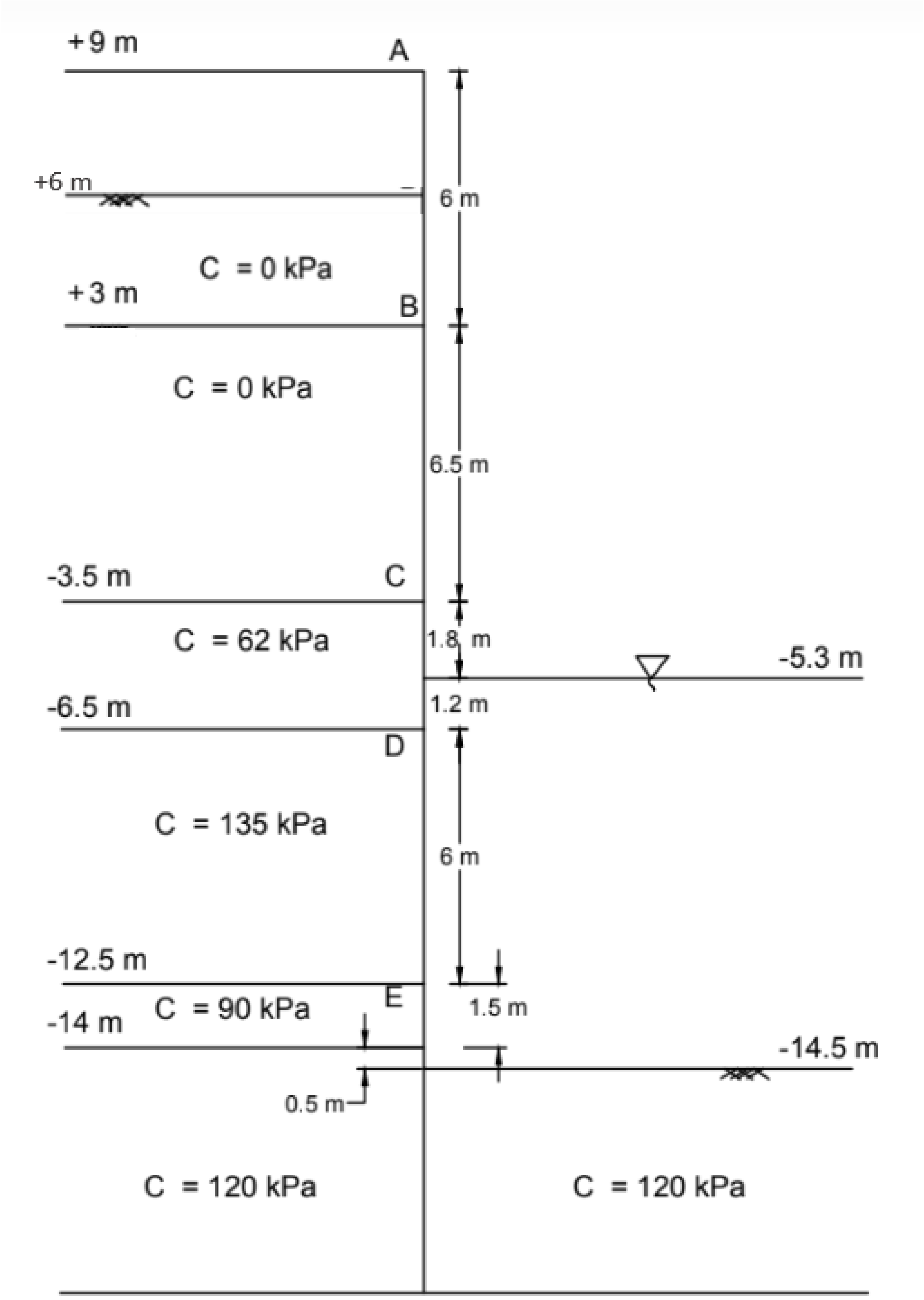


Figure: Soil Profile

ACTIVE AND PASSIVE EARTH PRESSURE CALCULATIONS

Earth pressure coefficients

Active earth pressure coefficient (k_a):

$k_a = \frac{1-sin(\varphi)}{1+sin(\varphi)}$

For φ = 0, $k_a = \frac{1-sin(0)}{1+sin(0)} = 1$

Passive earth pressure coefficient (k_p):

$K_p = \frac{1+sin(\varphi)}{1-sin(\varphi)}$

For φ = 0, $K_p = \frac{1+sin(0)}{1-sin(0)} = 1$

Active earth pressure (P_a)

Active earth pressures at different points are calculated below,

At A,

$P_a = K_a\gamma h + \gamma_w h_w - 2c\sqrt{K_a}$

$P_a = 0$

At B,

$P_a = K_a\gamma h + \gamma_w h_w - 2c\sqrt{K_a}$

$P_a = 0$

At C,

(just above)

$P_a = K_a\gamma h + \gamma_w h_w - 2c\sqrt{K_a}$

$P_a = (12 \times 6.5)$ = *78 kPa*

(just below)

$P_a = K_a\gamma h + \gamma_w h_w - 2c\sqrt{K_a}$

$P_a = (12 \times 6.5) - (2 \times 62) =$ *-46 kPa*

At E,

(just above)

$P_a = K_a\gamma h + \gamma_w h_w - 2c\sqrt{K_a}$

$P_a = (12 \times 6.5) + (3 \times 20) - (2 \times 62)$ = *14 kPa*

(just below)

$P_a = K_a\gamma h + \gamma_w h_w - 2c\sqrt{K_a}$

$P_a = (12 \times 6.5) + (3 \times 20) - (2 \times 135)$ = *-132 kPa*

At F,

(just above)

$P_a = K_a\gamma h + \gamma_w h_w - 2c\sqrt{K_a}$

$P_a = (12 \times 6.5) + (9 \times 20) - (2 \times 135)$ = *-12 kPa*

(just below)

$P_a = K_a\gamma h + \gamma_w h_w - 2c\sqrt{K_a}$

$P_a = (12 \times 6.5) + (9 \times 20) - (2 \times 90)$ = *78 kPa*

At G,

(just above)

$P_a = K_a\gamma h + \gamma_w h_w - 2c\sqrt{K_a}$

$P_a = (12 \times 6.5) + (10.5 \times 20) - (2 \times 90)$ = *108 kPa*

(just below)

$P_a = K_a\gamma h + \gamma_w h_w - 2c\sqrt{K_a}$

$P_a = (12 \times 6.5) + (10.5 \times 20) - (2 \times 120)$ = *48 kPa*

At H,

$P_a = K_a\gamma h + \gamma_w h_w - 2c\sqrt{K_a}$

P_a = (12 x 6.5) + (11 x 20) – (2 x 120) = *84 kPa*

At X,

P_a = (12 x 6.5) + (20 x (11 + y)) – (2 x 120)

P_a = 220 + 20y - 162

P_a = 20y + 58

Passive earth pressure (P_P)

Passive earth pressures at different points are calculated below,

At D,

P_p = 0

At H,

(Just above)

P_p = (9.81 x 9.2) = *90.25 kPa*

(Just below)

P_p = (9.81 x 9.2) + (2 x 120) = *330.25 kPa*

At X,

P_p = (9.81 x 9.2) + (2 x 120) + (20 x y)

P_p = *330.25 + 20y*

Active and passive moment calculations

Active moment

From active earth pressure diagram, moment about bottom point is

$M_a = \left(58 * y * \frac{y}{2}\right) + (0.5 * 20y * y * \left(\left(\frac{y}{3}\right)\right) + (48 * 0.5 * \left(y + \left(\frac{0.5}{2}\right)\right) + \left(0.5 * 10 * 0.5 * \left(y + \left(\frac{0.5}{3}\right)\right)\right) + \left(78 * 1.5 * \left(y + 0.5 + \left(\frac{1.5}{2}\right)\right)\right) + \left(0.5 * 30 * 1.5 * \left(y + 0.5 + \left(\frac{1.5}{3}\right)\right)\right) + (0.5 * 14 * 0.7 * \left(y + 8 + \left(\frac{0.7}{3}\right)\right) + (0.5 * 6.5 * 78 * (y + 11 + \left(\frac{6.5}{3}\right))$

$M_a = 3.333y^3 + 29y^2 + 24y + 6 + 2.5y + 0.417 + 117y + 146.25 + 22.5y + 22.5 + 4.9y + 40.343 + 253.5y + 3337.75$

$M_a = 3.333 y^3 + 29y^2 + 424.4y + 3553.26$

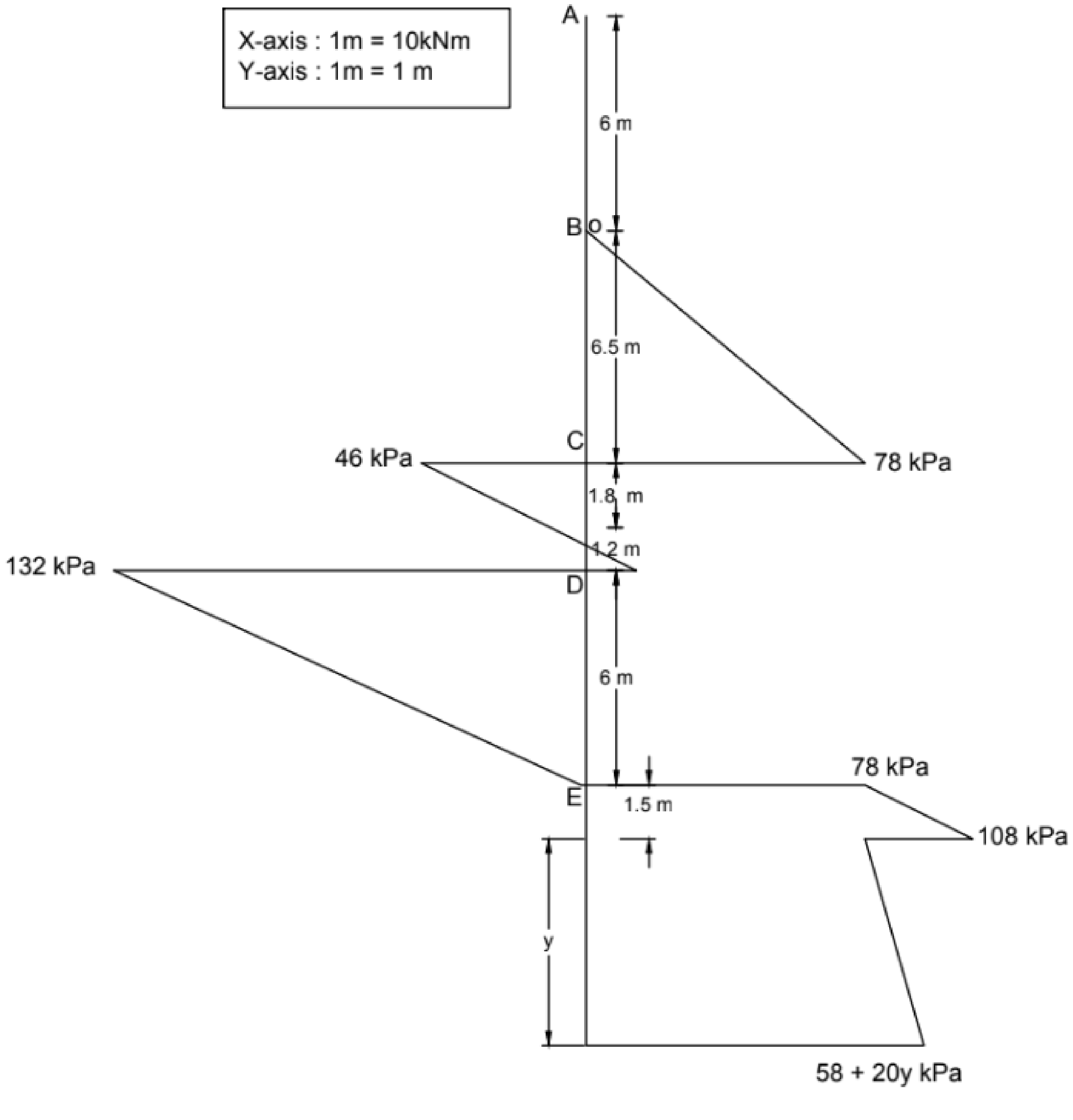


Figure: Active Earth Pressure Diagram

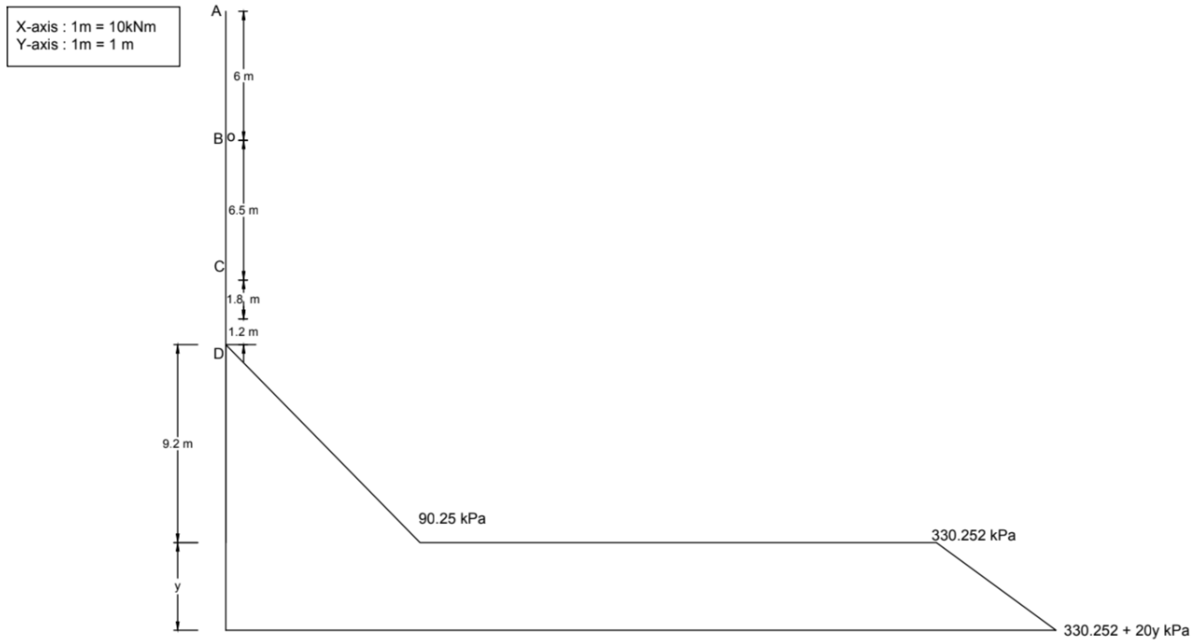


Figure: Passive Earth Pressure Diagram

Passive moment

From passive earth pressure diagram, moment about bottom point is

$M_p = \left(330.25 * \ y * \ \frac{y}{2}\right) + (0.5 * 20y * y * \left(\frac{y}{3}\right)) + \left(0.5 * 9.2 * 90.25 * \left(y + \left(\frac{9.2}{3}\right)\right)\right.$

$M_p = 3.333y^3 + 165.13y^2 + 415.15y + 1273.13$

Taking a factor of safety of 2 on passive side,

$\frac{M_p}{2} = 1.665y^3 + 82.56y^2 + 207.6y + 636.565$

Equating the moments on active and passive side,

$3.333\ y^3 + 29y^2 + 424.4y + 3553.26 = 1.665y^3 + 82.56y^2 + 207.6y + 636.565$

$y = 14.08$ m

Total Height of Coffer Dam (H):

H = y + Height of soil and water to be retained

H = 14 + 23.5

H = 37.5 m

Dimensions of Cofferdam

From IS code, cellular structures shall be checked against cell shear, sliding. Taking the diameter of cofferdam, as 10.19 m, the factor of safety values for various mechanisms are greater than the required values. Design of cofferdam calculations and check for stability against various parameters are given below.

CHECK FOR STABILITY

Factor of safety

Height of Coffer dam, H = 37.5 m

Taking diameter, D = 10.19 m

From *IS:9527 (part 4)*,

Effective Width of cell, B = 8.51 m

Sliding

$$\text{FOS} = \frac{Resisting\ force}{Sliding\ force}$$

$$\text{FOS} = \frac{\mu W + P_p}{P_a + 162.736}$$

Taking μ= tan ϕ = tan 30

$$W = \left(\frac{10+20}{2}\right) \times 8.51 \times 37.5$$

W = 4786.87 kN/unit width

$$\text{FOS} = \frac{(15\ x\ 8.51\ \ x\ 37.5)\tan 30 + 6998.578}{3196.4}$$

FOS = 3.05

Tilting/Overturning

FOS = $\frac{Resisting\ moment}{Overtruning\ moment}$

FOS = $\frac{M_p}{M_a}$

FOS = *2*

Borehole – D8

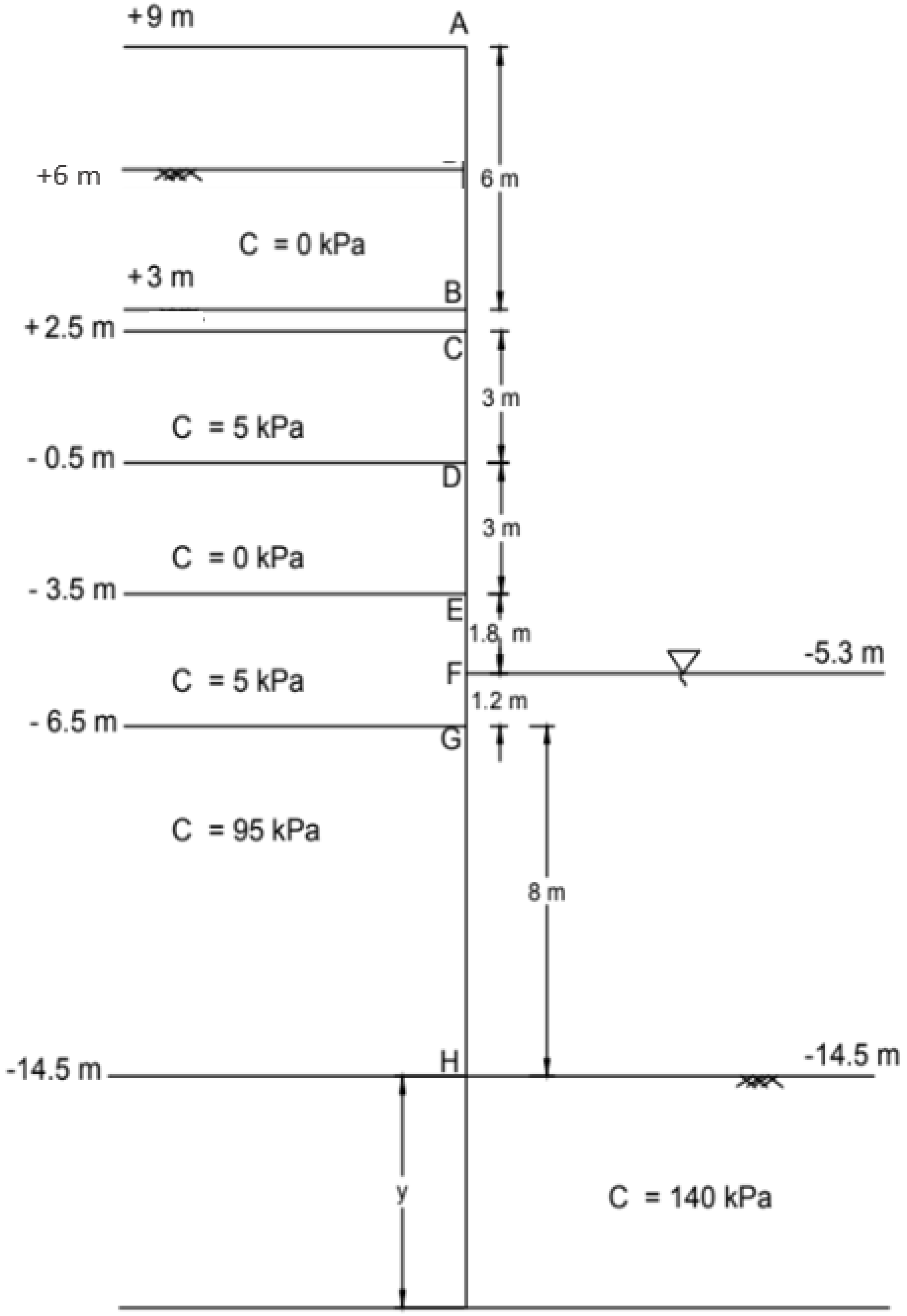


Figure: Soil Profile

ACTIVE AND PASSIVE EARTH PRESSURE CALCULATIONS

Earth pressure coefficients

Active earth pressure coefficient (k_a):

$k_a \quad = \quad \frac{1-sin(\varphi)}{1+sin(\varphi)}$

For φ = 0, $\quad k_a \quad = \quad \frac{1-sin(0)}{1+sin(0)} \quad = \quad 1$

Passive earth pressure coefficient (k_p):

$K_p \quad = \quad \frac{1+sin(\varphi)}{1-sin(\varphi)}$

For φ = 0, $\quad K_p \quad = \quad \frac{1+sin(0)}{1-sin(0)} \quad = \quad 1$

Active earth pressure (P_a)

Active earth pressures at different points are calculated below,

At A,

$P_a = K_a\gamma h + \gamma_w h_w - 2c\sqrt{K_a}$

$P_a = 0$

At B,

$P_a = K_a\gamma h + \gamma_w h_w - 2c\sqrt{K_a}$

$P_a = 0$

At C,

(just above)

$P_a = K_a\gamma h + \gamma_w h_w - 2c\sqrt{K_a}$

$P_a = (12 \times 0.5) = \mathit{6\ kPa}$

(just below)

$P_a = K_a\gamma h + \gamma_w h_w - 2c\sqrt{K_a}$

$P_a = (12 \times 0.5) - (2 \times 5) = \mathit{-4\ kPa}$

At D,

(just above)

$P_a = K_a\gamma h + \gamma_w h_w - 2c\sqrt{K_a}$

$P_a = (12 \times 3.5) - (2 \times 5)$ $= \mathit{32\ kPa}$

(just below)

$P_a = K_a\gamma h + \gamma_w h_w - 2c\sqrt{K_a}$

$P_a = (12 \times 3.5) = 42\ \mathit{kPa}$

At E,

(just above)

$P_a = K_a\gamma h + \gamma_w h_w - 2c\sqrt{K_a}$

$P_a = (12 \times 6.5) = \mathit{78\ kPa}$

(just below)

$P_a = K_a\gamma h + \gamma_w h_w - 2c\sqrt{K_a}$

$P_a = (12 \times 6.5) - (2 \times 5) = \mathit{68\ kPa}$

At G,

(just above)

$P_a = K_a\gamma h + \gamma_w h_w - 2c\sqrt{K_a}$

$P_a = (12 \times 9.5) - (2 \times 5) = \mathit{104\ kPa}$

(just below)

$P_a = K_a\gamma h + \gamma_w h_w - 2c\sqrt{K_a}$

$P_a = (12 \times 9.5) - (2 \times 95) = \mathit{-76\ kPa}$

At H,

(just above)

$P_a = K_a\gamma h + \gamma_w h_w - 2c\sqrt{K_a}$

P_a = (12 x 9.5) + (8 x 20) – (2 x 95) = *84 kPa*

(just below)

$P_a = K_a\gamma h + \gamma_w h_w - 2c\sqrt{K_a}$

P_a = (12 x 9.5) + (8 x 20) – (2 x 140) = *-6 kPa*

At X,

P_a = (12 x 9.5) + (8 x (20 + y)) – (2 x 140)

P_a = 114 + 160 + 20y - 280

$P_a = 20y - 6$

Passive earth pressure (P_P)

Passive earth pressures at different points are calculated below,

At F,

P_p = 0

At H,

(Just above)

P_p = (9.81 x 9.2) = *90.25 kPa*

(Just below)

P_p = (9.81 x 9.2) + (2 x 140) = *370.25 kPa*

At X,

P_p = (9.81 x 9.2) + (2 x 140) + (20 x y)

P_p = 370.25 + 20y

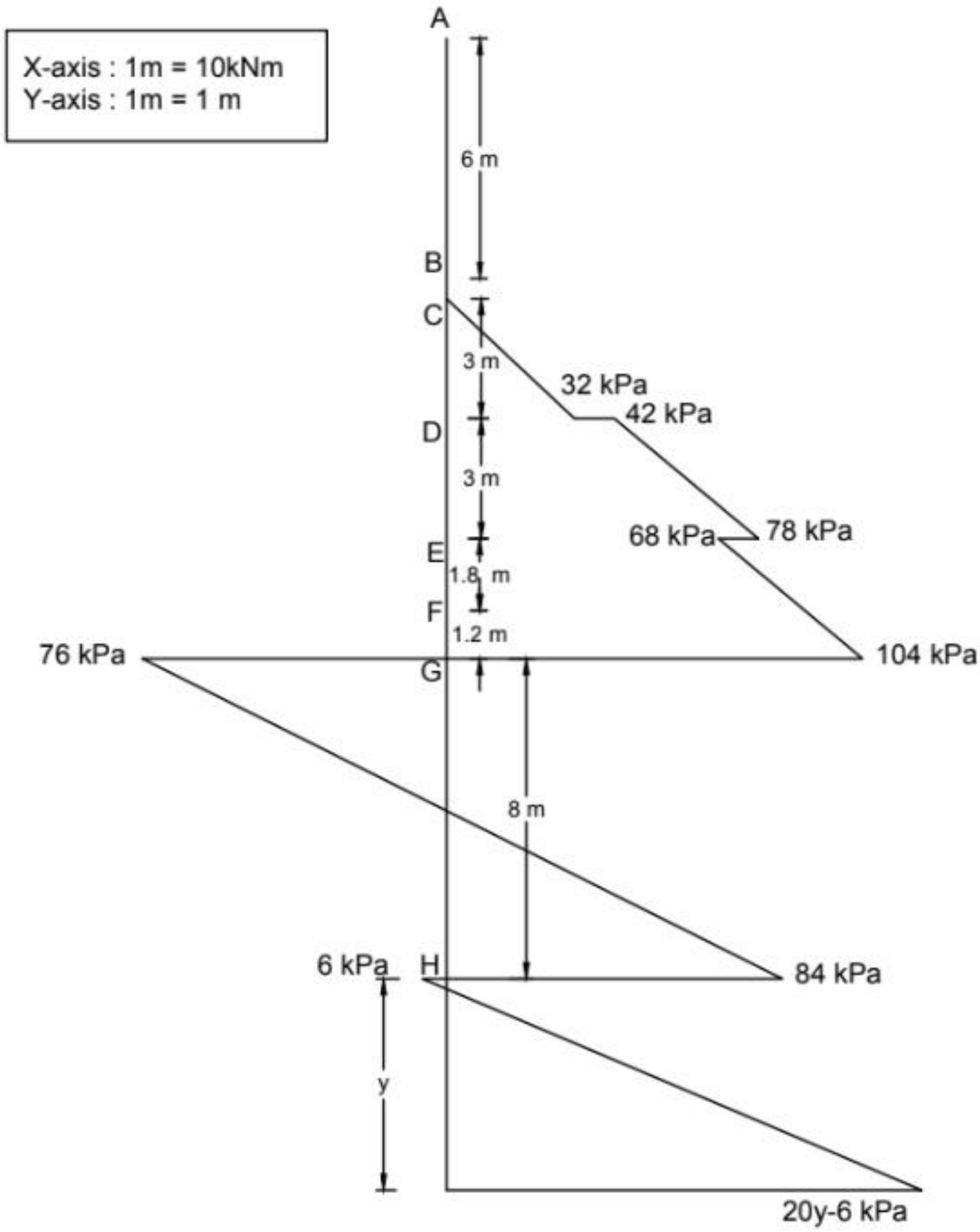


Figure: Active Earth Pressure Diagram

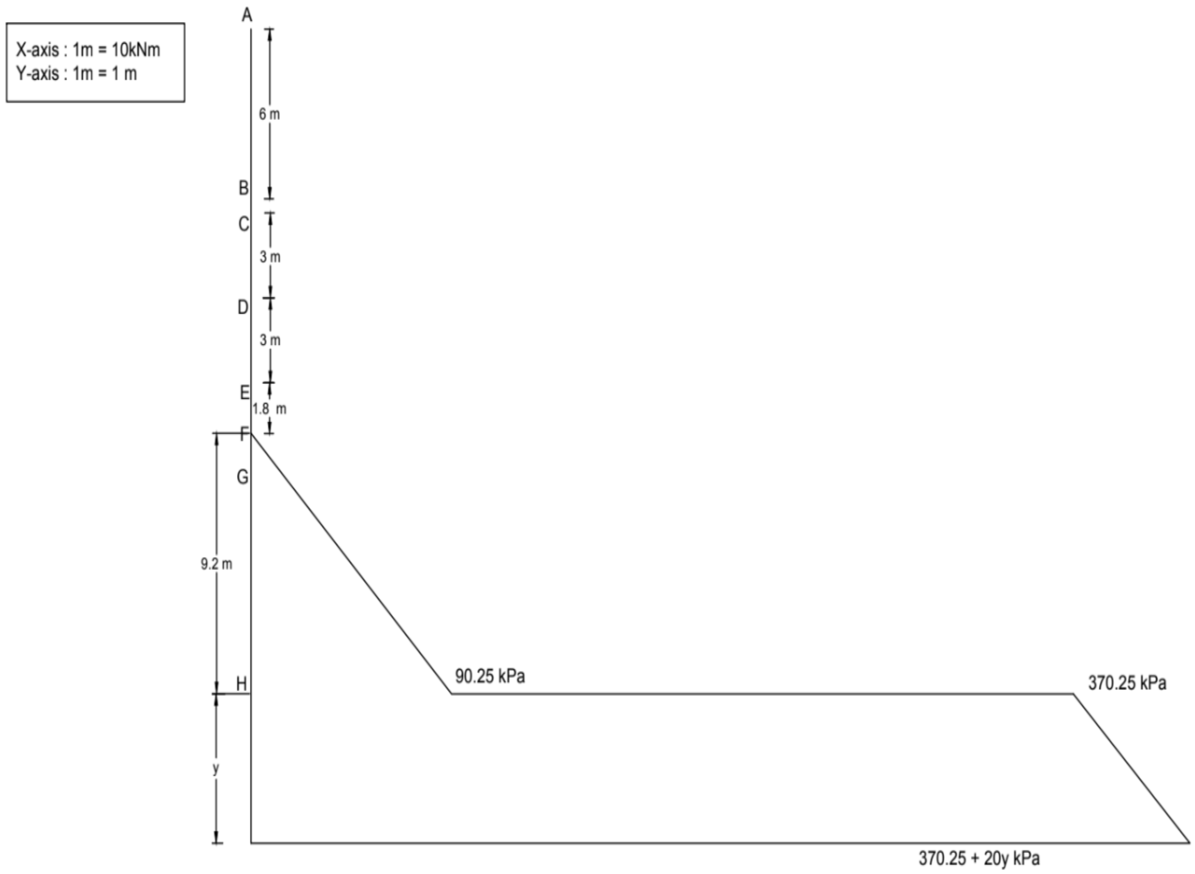


Figure: Passive Earth Pressure Diagram

Active and passive moment calculations

Active moment

From active earth pressure diagram, moment about point H is

$M_a = \left(0.5 * (20y - 6) * y * \frac{y}{3}\right) + (0.5 * 84 * 4.2 * \left(y + \left(\frac{4.2}{3}\right)\right) + (68 * 3 * \left(y + 8 + \left(\frac{3}{2}\right)\right) + \left(0.5 * 36 * 3\ (y + 8 + 1)\right) + \left(42 * 3 * (y + 11 + 1.5)\right) + \left(0.5 * 36 * 3\ (y + 11 + 1\ )\right) + (0.5 * 32 * 2.67 * \left(y + 14 + \left(\frac{2.67}{3}\right)\right) + (0.5 * 0.5 * 6\ (y + 17 + \left(\frac{0.5}{3}\right))$

$M_a = 3.333y^3 - y^2 + 176.4y + 246.96 + 204y + 1938 + 54y + 486 + 126y + 1575 + 54y + 648 + 42.72y + 636.06 + 1.5y + 25.75$

$M_a = 3.333\ y^3 - y^2 + 658.62y + 5555.77$

Passive moment

From passive earth pressure diagram, moment about point H is

M_p=$\left(370.25 * \ y * \ \frac{y}{2}\right) + (0.5 * 20y * y * \left(\frac{y}{3}\right)) + (0.5 * 9.2 * 90.25 * \left(y + \left(\frac{8}{3}\right)\right)$

$M_p = 3.333y^3 + 185.13y^2 + 415.15y + 1107.067$

Taking a factor of safety of 2 on passive side,

$\frac{M_p}{2} = 1.665y^3 + 92.56y^2 + 207.6y + 553.53$

Equating the moments on active and passive side,

$3.333\, y^3 - y^2 + 658.62y + 5555.77 = 1.665y^3 + 92.56y^2 + 207.6y + 553.53$

$y = 12.2$ m

Total Height of Coffer Dam (H):

H = y + Height of soil and water to be retained

H = 12.2 + 23.5

H = 36 m

Dimensions of Cofferdam

From IS code, cellular structures shall be checked against cell shear, sliding. Taking the diameter of cofferdam, as 10.19 m, the factor of safety values for various mechanisms are greater than the required values. Design of cofferdam calculations and check for stability against various parameters are given below.

CHECK FOR STABILITY

Factor of safety

Height of Coffer dam, H = 36 m

Taking diameter, D = 10.19 m

From *IS:9527 (part 4)*,

Effective Width of cell, B = 8.51 m

Sliding

FOS = $\frac{Resisting\ force}{Sliding\ force}$

FOS = $\frac{\mu W + P_p}{P_a + 162.736}$

Taking μ= tan ϕ = tan 30

W = $(\frac{10+20}{2})$ x 8.51 x 36

W = 4595.4 kN/unit width

FOS = $\frac{(15\ x\ 8.51\ \ x\ 36)\tan 30 + 6605.775}{2183.62}$

FOS = 4.24

Tilting/Overturning

FOS = $\frac{Resisting\ moment}{Overtruning\ moment}$

FOS = $\frac{M_p}{M_a}$

FOS = 2

ENCLOSURE 4

Design and Stability Calculations for Circular Cellular Cofferdam

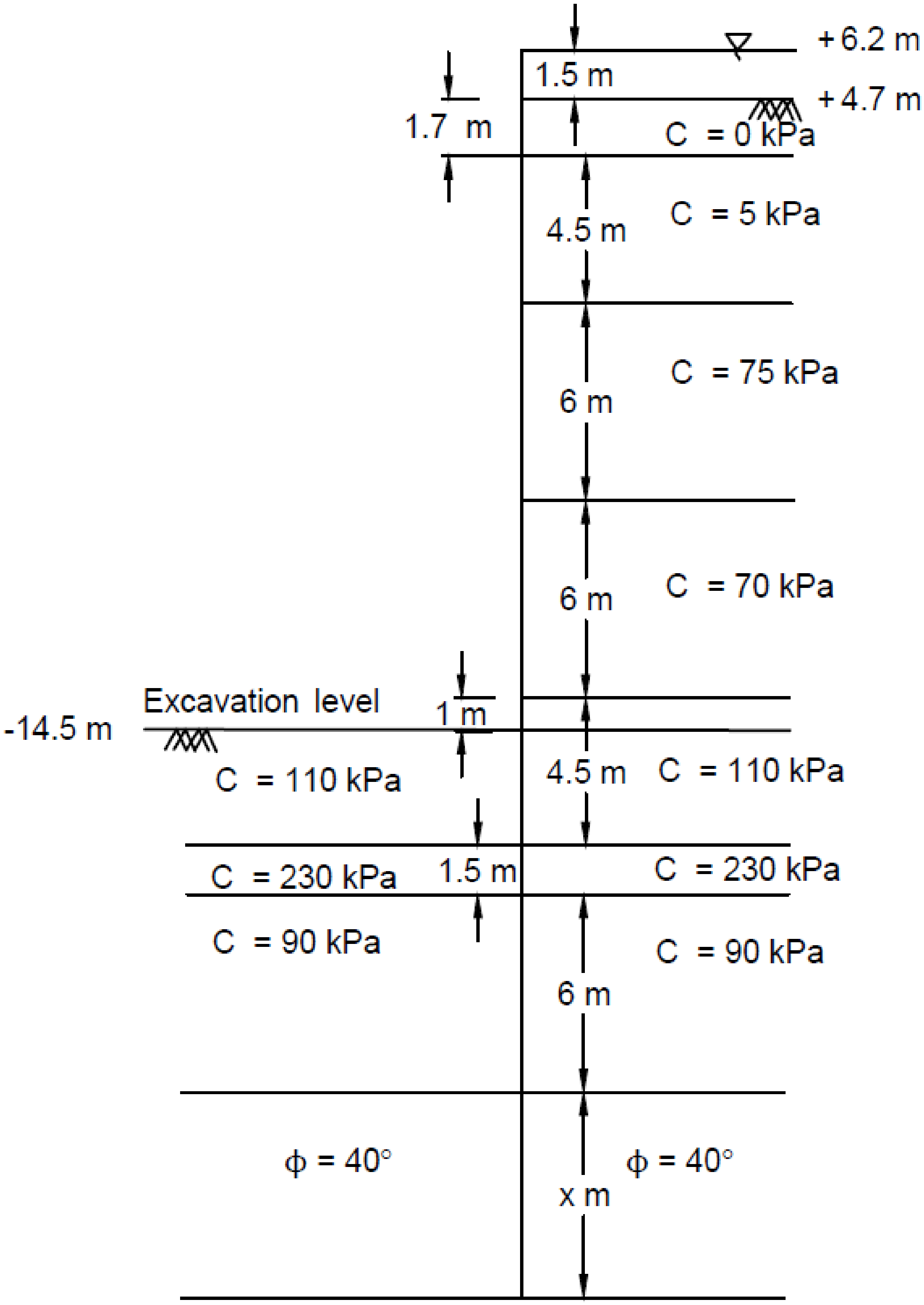


Figure A2. 1: Soil Profile

ACTIVE AND PASSIVE EARTH PRESSURE CALCULATIONS

Earth pressure coefficients

Active earth pressure coefficient (k_a):

$k_a \quad = \quad \frac{1-sin(\varphi)}{1+sin(\varphi)}$

For φ = 0, $\quad k_a \quad = \quad \frac{1-sin(0)}{1+sin(0)} \quad = \quad 1$

For $\varphi = 40^o$, $\quad k_a \quad = \quad \frac{1-sin(40)}{1+sin(40)} \quad = \quad 0.22$

Passive earth pressure coefficient (k_p):

$K_p \quad = \quad \frac{1+sin(\varphi)}{1-sin(\varphi)}$

For φ = 0, $\quad K_p \quad = \quad \frac{1+sin(0)}{1-sin(0)} \quad = \quad 1$

For $\varphi = 40^o$, $\quad K_p \quad = \quad \frac{1+\sin\ (40)}{1-sin(40)} \quad = \quad 4.6$

Active earth pressure (P_a)

Active earth pressures at different points are calculated below,

At A,

$P_a = K_a\gamma h + \gamma_w h_w - 2c\sqrt{K_a}$

$P_a = 0$

At B,

$P_a = K_a\gamma h + \gamma_w h_w - 2c\sqrt{K_a}$

P_a = 3.2 x 10 = *32 kPa*

At C,

(just above)

$P_a = K_a\gamma h + \gamma_w h_w - 2c\sqrt{K_a}$

P_a = (3.2 x 10) +(12 x 4.5) -(2 x 5) = *76 kPa*

(just below)

$P_a = K_a\gamma h + \gamma_w h_w - 2c\sqrt{K_a}$

P_a = (3.2 x 10) +(12 x 4.5) -(2 x 75) = *-64 kPa*

At D,

(just above)

$P_a = K_a\gamma h + \gamma_w h_w - 2c\sqrt{K_a}$

P_a = (3.2 x 10) +(12 x 4.5) +(20 x 6)) -(2 x 75) = *56 kPa*

(just below)

$P_a = K_a\gamma h + \gamma_w h_w - 2c\sqrt{K_a}$

$P_a = (3.2 \times 10) + (12 \times 4.5) + (20 \times 6) - (2 \times 70) \quad = \mathit{66\ kPa}$

At E,

(just above)

$P_a = K_a\gamma h + \gamma_w h_w - 2c\sqrt{K_a}$

$P_a = (3.2 \times 10) + (12 \times 4.5) + (20 \times 12) - (2 \times 70) = \mathit{186\ kPa}$

(just below)

$P_a = K_a\gamma h + \gamma_w h_w - 2c\sqrt{K_a}$

$P_a = (3.2 \times 10) + (12 \times 4.5) + (20 \times 12) - (2 \times 110) = \mathit{106\ kPa}$

At F,

$P_a = K_a\gamma h + \gamma_w h_w - 2c\sqrt{K_a}$

$P_a = (3.2 \times 10) + (12 \times 4.5) + (20 \times 13) - (2 \times 110) = \mathit{126\ kPa}$

At G,

(just above)

$P_a = K_a\gamma h + \gamma_w h_w - 2c\sqrt{K_a}$

$P_a = (3.2 \times 10) + (12 \times 4.5) + (20 \times 16.5) - (2 \times 110) = \mathit{196\ kPa}$

(just below)

$P_a = K_a\gamma h + \gamma_w h_w - 2c\sqrt{K_a}$

$P_a = (3.2 \times 10) + (12 \times 4.5) + (20 \times 16.5) - (2 \times 230) = \mathit{-44\ kPa}$

At H,

(just above)

$P_a = K_a\gamma h + \gamma_w h_w - 2c\sqrt{K_a}$

$P_a = (3.2 \times 10) + (12 \times 4.5) + (20 \times 18) - (2 \times 230) = \mathit{-14\ kPa}$

(just below)

$P_a = K_a\gamma h + \gamma_w h_w - 2c\sqrt{K_a}$

$P_a = (3.2 \times 10) + (12 \times 4.5) + (20 \times 18) - (2 \times 90) = \mathit{266\ kPa}$

At I,

(just above)

$P_a = K_a\gamma h + \gamma_w h_w - 2c\sqrt{K_a}$

$P_a = (3.2 \times 10) + (12 \times 4.5) + (20 \times 24) - (2 \times 90) = \mathit{386\ kPa}$

(just below)

$P_a = K_a\gamma h + \gamma_w h_w - 2c\sqrt{K_a}$

$P_a = (31.7 \times 10) + (0.222 \times ((2 \times 4.5) + (10 \times 24)) =$ *372.3 kPa*

At X,

$P_a = (10 \times (31.7 + X)) + (0.222 \times ((2 \times 4.5) + (10 \times (24 + X))$

$P_a = 317 + 10X + 1.998 + 52.8 + 2.2X$

$P_a = 371.8 + 12.2X$

Passive earth pressure (P_P)

Passive earth pressures at different points are calculated below,

At F,

$P_p = 2 \times 110 \times \sqrt{1}$ = *220 kPa*

At G,

(Just above)

$P_p = (3.5 \times 20) + (2 \times 110 \times \sqrt{1})$ = *290 kPa*

(Just below)

$P_p = (3.5 \times 20) + (2 \times 230)$ = *530 kPa*

At H,

(Just above)

$P_p = (5 \times 20) + (2 \times 230)$ = *560 kPa*

(Just below)

$P_p = (5 \times 20) + (2 \times 90)$ = *280 kPa*

At I,

(Just above)

$P_p = (11 \times 20) + (2 \times 90)$ = *400 kPa*

(Just below)

$P_p = (11 \times 10) + (4.6 \times 10 \times 11)$ = *616 kPa*

At X,

$P_p = (10 \times (11 + X) + (4.6 \times (11 + X) \times 10)$

$P_p = 110 + 10X + 506 + 46X$

$P_p = 616 + 56X$

Active and passive moment calculations:

Active moment

From active earth pressure diagram, moment about point H is

$M_a = (372 \times X \times \frac{X}{2}) + (\frac{1}{2} \times 12.2X \times X \times \frac{X}{3}) + (266 \times 6 \times (X + \frac{6}{2})) + (\frac{1}{2} \times 120 \times 6 \times (X + \frac{6}{3})) + (106 \times 4.5 \times (X + 7.5 + \frac{4.5}{2})) + (\frac{1}{2} \times 90 \times 4.5 \times (X + 7.5 + \frac{4.5}{3})) + (66 \times 6 \times (X + 12 + \frac{6}{2})) + (\frac{1}{2} \times 120 \times 6 \times (X + 12 + \frac{6}{3})) + (\frac{1}{2} \times 2.8 \times 56 \times (X + 18 + \frac{2.8}{3})) + (32 \times 4.5 \times (X + 24 + \frac{4.5}{2})) + (\frac{1}{2} \times 44 \times 4.5 \times (X + 24 + \frac{4.5}{3})) + (\frac{1}{2} \times 32 \times 3.2 \times (X + 28.5 + \frac{3.2}{3}))$

$M_a = 186 X^2 + 2.04 X^3 + 1596 X + 4788 + 360 X + 720 + 477 X + 4650.75 + 202.5 X + 1822.5 + 396 X + 5940 + 360 X + 5040 + 78.4 X + 1484.37 + 144 X + 3780 + 99 X + 2524.5 + 51.2 X + 1513.8$

$M_a = 2.04 X^3 + 186 X^2 + 3764.1 X + 32263.92$

Passive moment

From passive earth pressure diagram, moment about point H is

$M_p = (616 \times X \times \frac{X}{2}) + (\frac{1}{2} \times 56X \times X \times \frac{X}{3}) + (280 \times 6 \times (X + \frac{6}{2})) + (\frac{1}{2} \times 120 \times 6 \times (X + \frac{6}{3})) + (530 \times 1.5 \times (X + 6 + \frac{1.5}{2})) + (\frac{1}{2} \times 30 \times 1.5 \times (X + 6 + \frac{1.5}{3})) + (220 \times 3.5 \times (X + 7.5 + \frac{3.5}{2})) + (\frac{1}{2} \times 70 \times 3.5 \times (X + 7.5 + \frac{3.5}{3}))$

$M_p = 308 X^2 + 9.333 X^3 + 1680 X + 5040 + 360 X + 720 + 795 X + 5366.25 + 22.5 X + 146.25 + 770 X + 7112.5 + 122.5 X + 1061.67$

$M_p = 9.333 X^3 + 308 X^2 + 3750 X + 19446.67$

Taking a factor of safety of 2 on passive side,

$\frac{M_p}{2} = 4.665 X^3 + 154 X^2 + 1875 X + 9723.335$

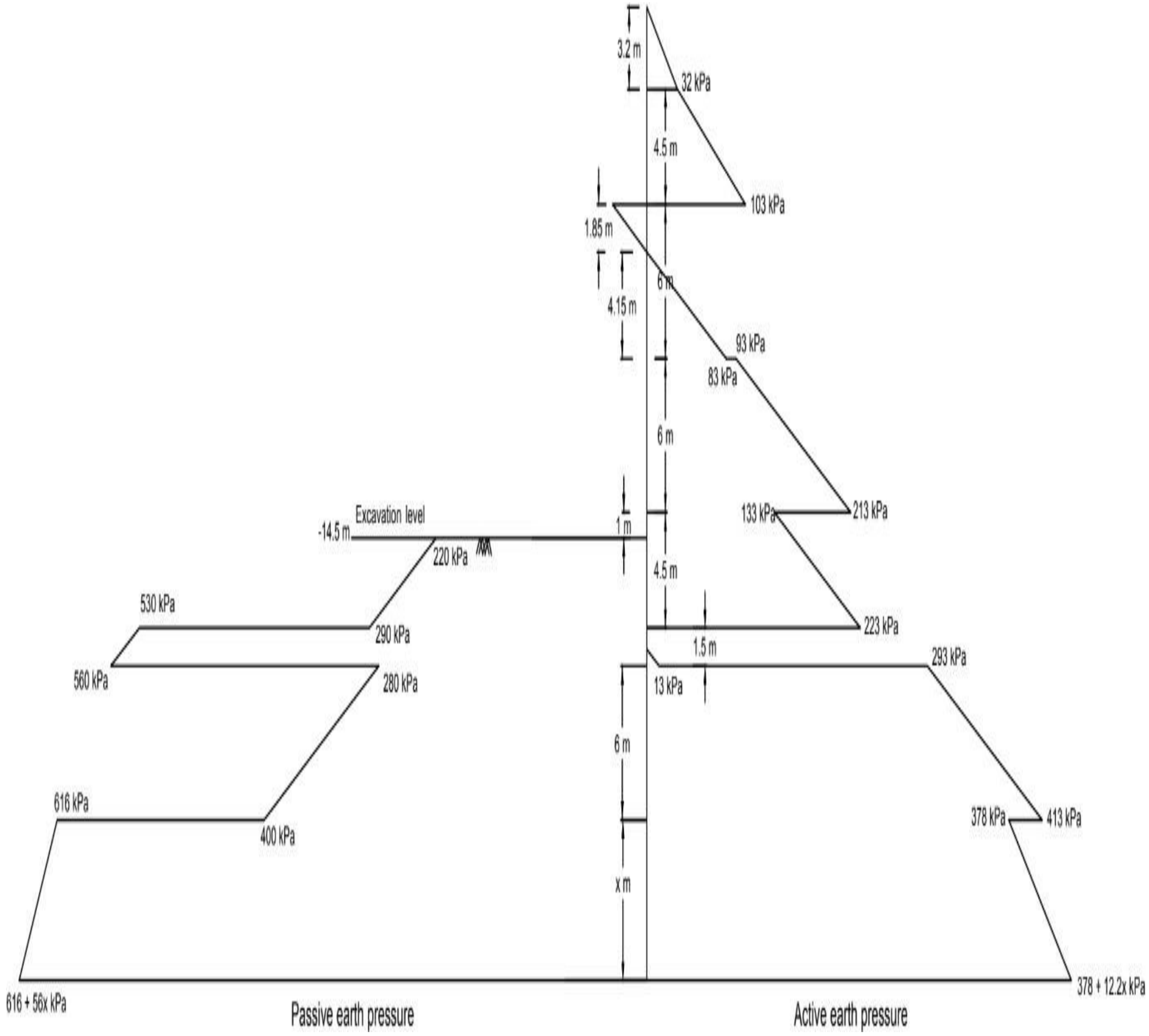


Figure A2.2: Active and Passive Earth Pressure Diagram

Equating the moments on active and passive side,

$2.04\ X^3 + 186\ X^2 + 3764.1\ X + 32263.92 = 4.665\ X^3 + 154\ X^2 + 1875\ X + 9723.335$

X = 37.49 m

Therefore, Total Depth of penetration of Coffer dam = 37.49 + 12

D = 49.5 m

Total Height of Coffer Dam (H):

H = D + Height of soil and water to be retained

H = 49.5 + 17.5 + 3.5

H = 70.2 m

CHECK FOR STABILITY

Factor of safety

Height of Coffer dam, H = 70.2 m

Taking diameter, D = 23.95 m

From *IS: 9527 (part 4)*,

Effective Width of cell, B = 19.91 m

Sliding

$$FOS = \frac{Resisting\ force}{Sliding\ force}$$

$$FOS = \frac{\mu W + P_p}{P_a + 162.736}$$

Taking $\mu = \tan\phi = \tan 30$

$$P_p = (\frac{220 + 290}{2} \text{ x } 3.5) + (\frac{530 + 560}{2} \text{ x } 1.5) + (\frac{280 + 400}{2} \text{ x } 6) + (\frac{616 + 2715.44}{2} \text{ x } 37.49)$$

P_p = 66197.84 kN/unit width

$$P_a = (\frac{1}{2} \text{ x } 32 \text{ x } 3.2) + (\frac{32 + 76}{2} \text{ x } 4.5) + (\frac{1}{2} \text{ x } 2.8 \text{ x } 56) + (\frac{66 + 186}{2} \text{ x } 6) + (\frac{106 + 196}{2} \text{ x } 4.5) + (\frac{266 + 386}{2} \text{ x } 6) + (\frac{372 + 829.378}{2} \text{ x } 37.49)$$

P_a = 26283.93 kN/unit width

$$W = (\frac{10+20}{2}) \text{ x } 70.2 \text{ x } 19.91$$

W = 20965.23 kN/unit width

$$FOS = \frac{66197.84 + ((20965.23)\ (\tan 30))}{26283.93}$$

FOS = 2.97

Tilting/Overturning

$$FOS = \frac{Resisting\ moment}{Overtruning\ moment}$$

$$FOS = \frac{M_p}{M_a}$$

FOS = *2*

Hence OK