Interim Report on

Geotechnical and Structural Design Aspects of Spillway - Kalpasar Dyke

as part of

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

Sanction order no. MoES/NCCR/44/Kalpasar/Spillway/2022

funded by

National Centre for Coastal Research
Ministry of Earth Sciences

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August 2022

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INTRODUCTION

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

The spillway is located in intertidal region of Dahej and is of 2.2 km long with 100 spans. There is only one borehole (L1) in the entire spillway location. According to borehole L1, the top 3 m is stiff clay followed by soft to very soft clay for 3 m and the rest is very stiff clay up to termination level i.e., 15 m. The additional soil investigations consists of 13 SPTs and 23 ECPTs and are scheduled to start in the first week of September, 2022. The preliminary geotechnical design has been carried out.The levels of various components of spillway are considered based on the detailed hydraulic analysis carried out by Prof. B. S. Murty. Spillway consists of ogee spillway, stilling basin, approach channel (from reservoir to the spillway) and spill channel (from spillway to the sea). This report deals with geotechnical and structural designs of spillway.

OVERVIEW and RESULTS

- For construction of spillway in dry condition, circular cellular cofferdams are proposed. The design of the same has been carried out and presented.
- The slope stability analyses for approach and spill channels are carried out with and slopes of 1 V: 2 H are recommended.
- The carriageway across the spillway needs to be supported by piles and recommendations are given on pile capacities for piles of varying lengths and diameter.
- A detailed study has been conducted on various slope protection and

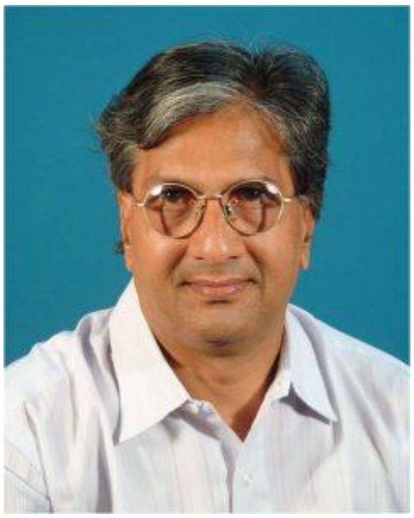
channel linings and suitable options are recommended.

- The structural design of spillway crest, breast wall, baffle blocks and end sill has been carried out.

RECOMMENDATIONS

A slope of 1 V : 2 H has to be adopted for both approach and spill channels. The height of circular cellular cofferdam is determined as 30.5 m with a diameter of 10.19 m. The excavation has to be done leaving 8 m from the cofferdam in order to utilize the passive resistance in front of the cofferdam. The flexible lining such as geotextile grout filled mattress and sand filled tubular mattress is recommended for both approach and spill channels. The capacities of piles carrying load from piers are determined for various lengths and diameters. The structural design of ogee weir crest, breast wall, baffle wall and end sill is presented in this report.

TEAM



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1. Preamble

The Kalpasar Department, Government of Gujarat, proposed to construct a multipurpose mega project comprising the construction of a dam for a length of about 60 km. Out of which, 30 km is in the gulf region and balance 30 km length is extended on both flanks up to nearest road crossing between Bhavnagar on the western coast and Dahej on the eastern coast of Gulf of Khambhat.

The proposed spillway is located in the intertidal region of Dahej. The length of the spillway is 2.2 km. Figure 1 shows spillway location on a google map image. Borehole L1 is the only one borehole available in the entire spillway location (enclosure 1), investigated up to a depth of 15.5 m from existing ground level (EGL) i.e., + 2.9 m by Coastal Marine Construction & Engineering Limited (COMACOE) soil investigation agency. Soil strata consist of stiff to very stiff silty clay up to 3 m followed by 3 m thick soft to very soft silty clay, and very stiff to hard silty clay up to 15.5 m.

The spillway discharges surplus water from the reservoir to the sea. The width of the spillway is 2200 m with center-to-center distance between piers is 22 m. With this, there will be 100 bays. The proposed type of gate is a vertical lift gate with a width of 18 m.

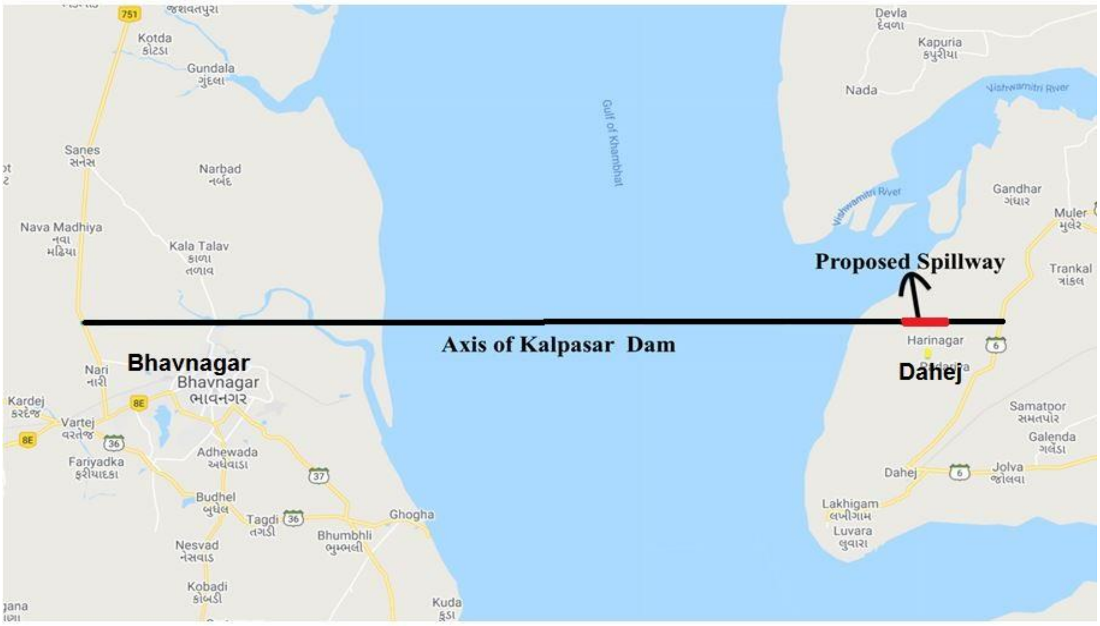


Figure 1. Location of spillway on a google map image

This report deals with the design of cellular circular cofferdam and pile foundation below piers, slope stability analysis of approach and spill channels, recommendations on suitable materials for channel lining and slope protection works, structural design of ogee weir crest, breast wall, baffle blocks and end sill.

2. Cofferdam

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. Earthen cofferdam, Rockfill 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, 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.

2.1 Cellular Cofferdam

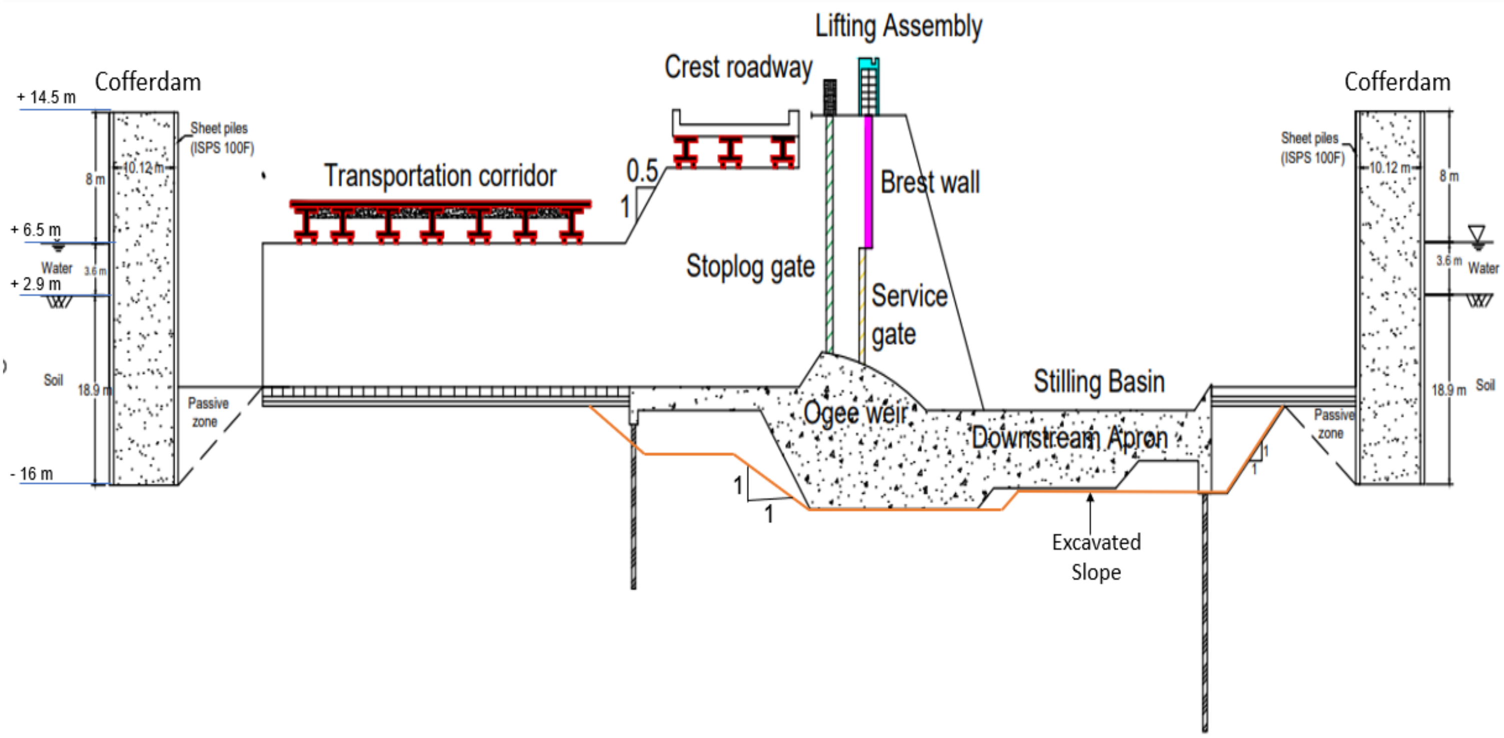


Figure 2. Cross section of spillway and cofferdam

A cellular cofferdam comprises interconnected cells that form a watertight 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 high interlock tension between adjacent cells.

Figure 2 shows the typical cross-section of spillway and cofferdams on the upstream and downstream sides.

2.2 Design of Circular Cellular Cofferdam

2.2.1 Soil profile

Cohesion values are interpreted from CIRIA that correlates the plasticity index to C_u/N where N is the SPT value (given in enclosure 2). The existing high tide level in the area is + 6.5 m w.r.t MSL. To get assistance from the passive zone and make an economical design, the depth of excavation immediately after the cofferdam is not needed to be the final excavation level. Figure 3 shows the soil profile.

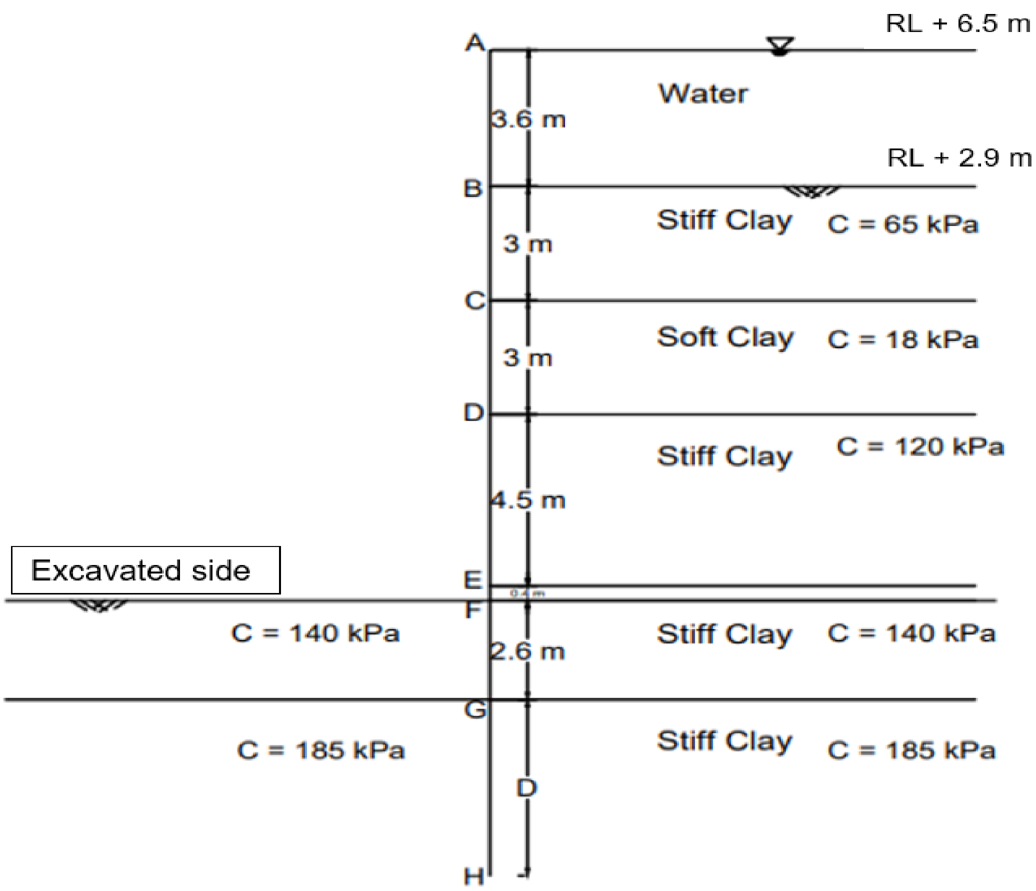


Figure 3. Soil profile

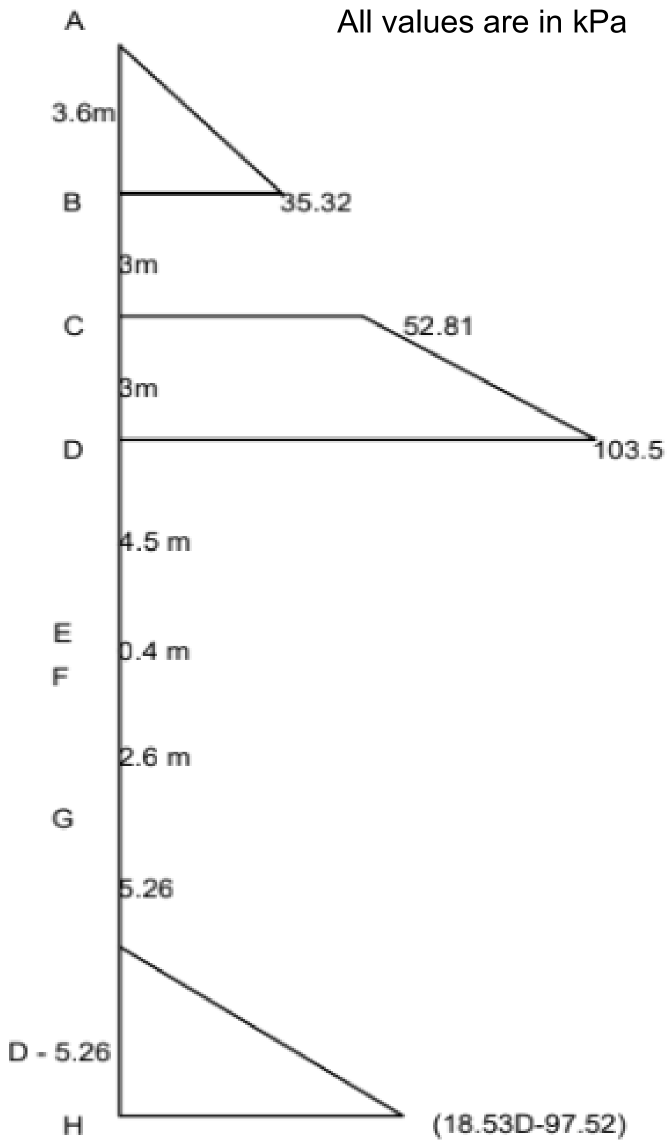


Figure 4. Active earth pressure diagram

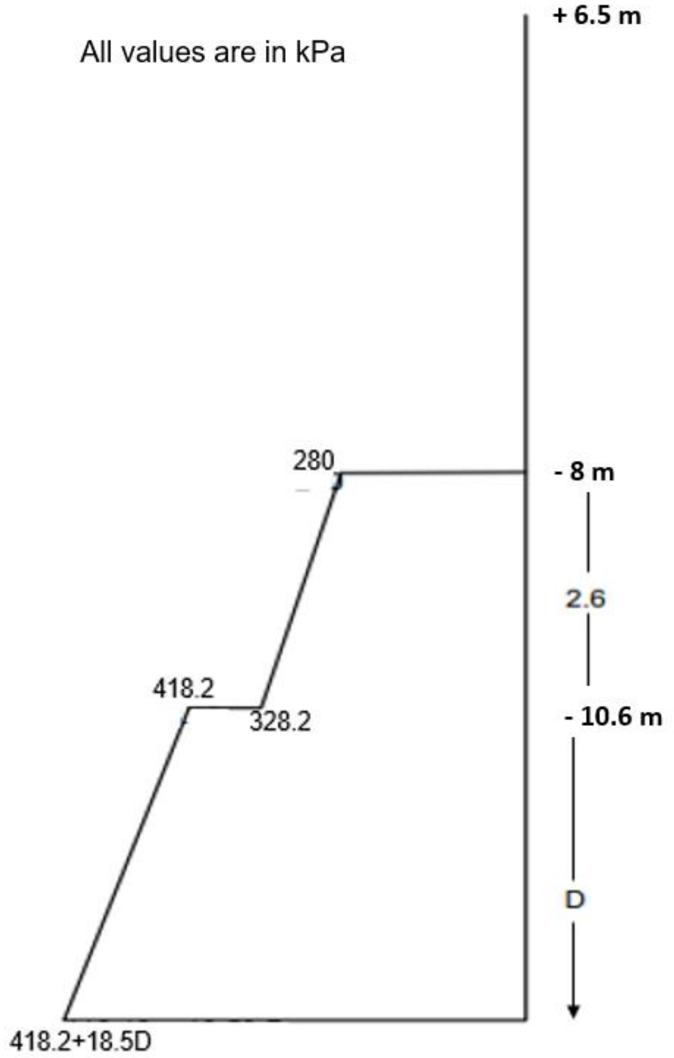


Figure 5. Passive earth pressure diagram

From active and passive pressure diagrams (shown in Figures 4 and 5), moments are calculated about the base taking a factor of safety of 2 on the passive side and the required depth of penetration of the cofferdam into the clay layer is worked out to be 7.7 m. Active and passive earth pressures and moment calculations are given in enclosure 3. Considering the freeboard as 1.5 times the height of the wave (according to IS: 10084 (part 1) -1982), the total height of the cofferdam is 30.5 m.

2.2.2 Dimensions of Cofferdam

From IS code, cellular structures shall be checked against cell shear, sliding, tilting, bursting of cell and bearing capacity (in clays). Taking the diameter of cofferdam, **as 10.19 m**, the factor of safety values for various mechanisms are greater than the required values (Table 1). Design of cofferdam calculations and check for stability against various parameters are given in enclosure 4.

Table 1: Factor of safety for various parameters

Mechanism	Required FOS	Obtained FOS
Cell Shear	1.25	2
Sliding	1.25	18
Tilting	1.2	2
Bursting of Cell	1.5	2.2
Bearing Capacity	1.5	1.76

From IS: 9527 (part 4*)*, the dimensions of each cell for the corresponding diameter of 10.19 m are given in Table 2. The plan of the cofferdam showing all dimensions is given in figure 6.

Table 2. Design parameters for cofferdam design

S. No	Design parameters	Dimensions (m)
1	Effective Width of cell (B)	8.51
2	Radius of Connecting Cells	3.06
3	Edge to edge distance between main cells	1.34
4	Center to center distance between main cells	11.53
5	Number of piles in cell	80

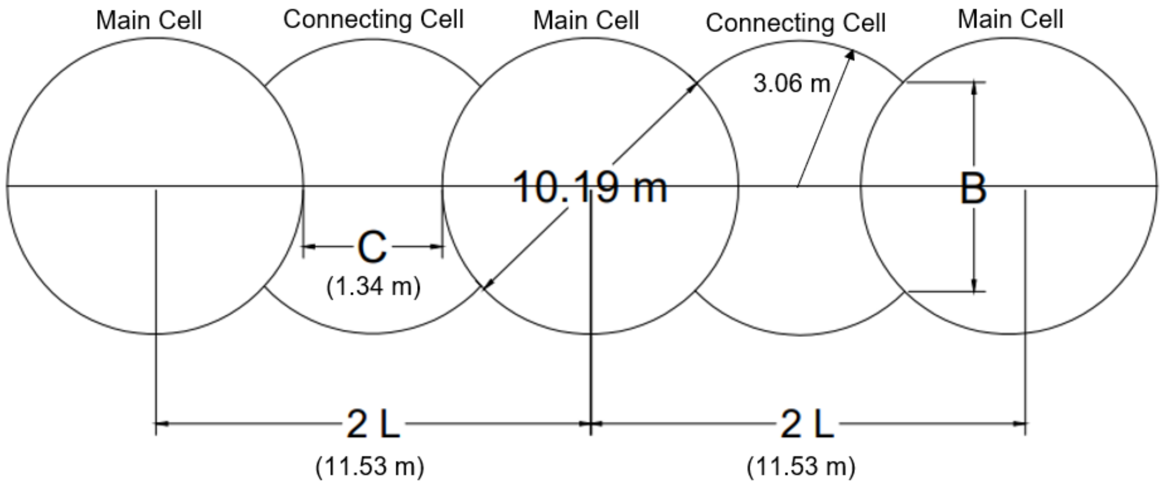
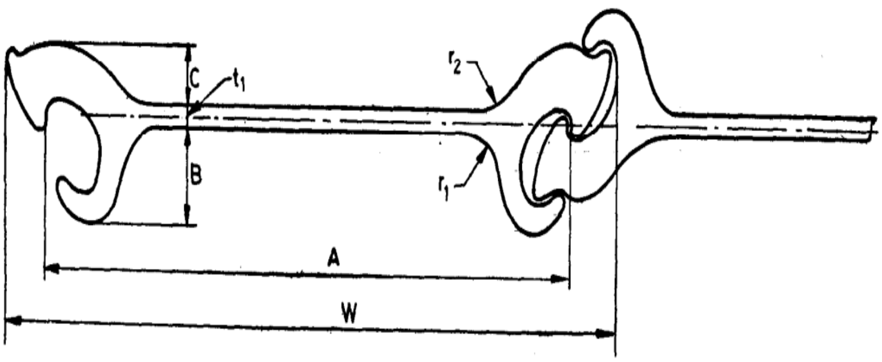


Figure 6. Plan of cellular cofferdam

2.2.3 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 piling section is given in figure 7.



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 7. Flat type piling section

2.2.4 Location of Cofferdam

To develop passive resistance in front of the cofferdam the required excavation has to be started after 8 m from the cofferdam. The cross section of the cofferdam is shown in figure 8.

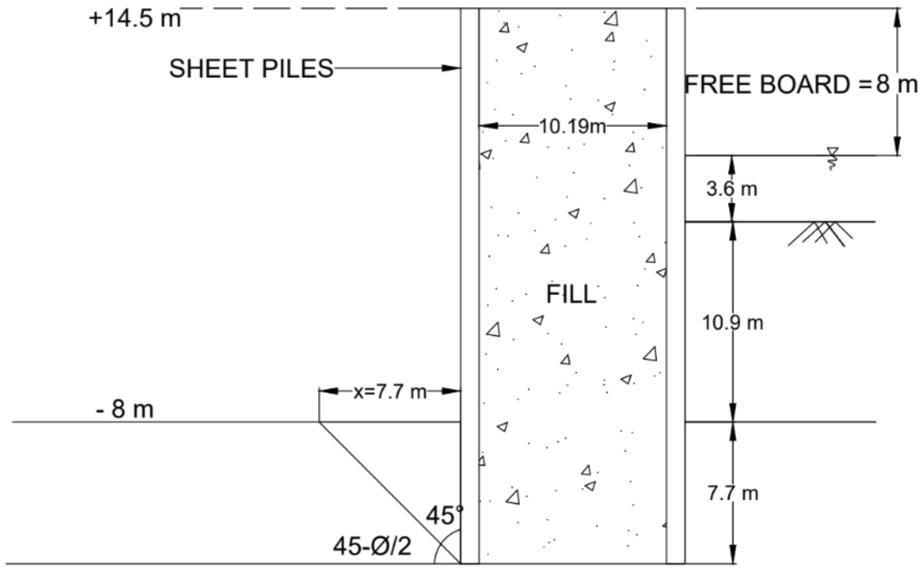


Figure 8. Cross section of the cofferdam

All the above calculations of cofferdam design have to be revised once additional soil investigation data is made available.

3 Slope stability analysis of approach and spill channels

3.1 Soil stratification and hydraulic levels

The water levels for both approach and spill channels are given in Tables 3 and 4.

Table 3: Hydraulic levels for approach channel

S. No	Hydraulic levels	Level (m)
1	Full Reservoir Level (FRL)	+ 3
2	High Flood Level (HFL)	+ 5
3	Dead Storage Level (DSL)	- 4

Table 4: Hydraulic levels for spill channel

S. No	Hydraulic levels	Level (m)
1	High Tide Level (HTL)	+ 6.5
2	Low Tide Level (LTL)	- 6.5

The channel bed of the approach channel is - 7 m and the spill channel is - 10 m for 1 km length and then - 8 m till the end.

3.2 Interpreted soil properties

Based on the L1 bore log information shear strength parameters (Table 5) are considered. Bulk unit weight and saturated unit weight of silty clay are assumed as 17 kN/m^3 and 20 kN/m^3 respectively. Unit weight of water is 10 kN/m^3

Slope stability analysis of approach and spill channels for 1 V: 2 H slope based on reported and shear strength parameters obtained from CIRIA correlation is carried out using Oasys Slope 19.0.

Table 5. Shear strength parameters considered for slope stability analysis

Depth (m)	Thickness (m)	SPT N	Reported Cohesion Cu (kPa)	CIRIA Cohesion Cu (kPa)	Reported Φ (⁰)	Soil Strata
0 to 3	3	14	48	64.7	7	Very stiff- Stiff silty clay
3 to 6	3	4	18	18	0	Soft – Very soft silty clay
6 to 10.5	4.5	26	53	122	12	Very stiff silty clay
10.5 to 13.5	3	26	53	144	12	Very stiff silty clay
13.5 to 15.5	2	39	66	185	12	Hard – Very stiff silty clay
>15.5*	-	39	71	185	12	Very stiff silty clay

* Borehole was terminated at 15.5 m. It is assumed that the same soil strata is continued up to pile termination level.

The analysis is carried out for the following water level conditions.

i. Water level at + 3 m (FRL) w.r.t MSL of approach channel
ii. Water level at - 4 m (DSL) w.r.t MSL of approach channel
iii. Water level at - 6.5 m (LTL) w.r.t MSL of spill channel

The analysis is carried out for various slopes and the slope yielding the minimum factor of safety is 1 V: 2 H. The schematic diagram and slip circles for different water level conditions of the recommended slope of approach and spills channels with reported and interpreted shear strength parameters are given in enclosure 5.

3.3 Results

The factor of safety of a slope is the ratio of the resisting moment to the driving moment. The slope is said to be stable if the safety factor is greater than or equal to 1.3 (as per Appendix A of IS 7894: Code of practice for stability analysis of earth dams).

Based on the L1 bore log data, slope stability analysis of the approach channel and spill channel is designed. The factor of safety values for the slope of 1 V: 2 H at different water level conditions for the two cases are given in Table 6. The slip circle corresponding to the minimum factor of safety for the case where the water level is at dead storage level (- 4 m) for the approach channel is shown in figure 9.

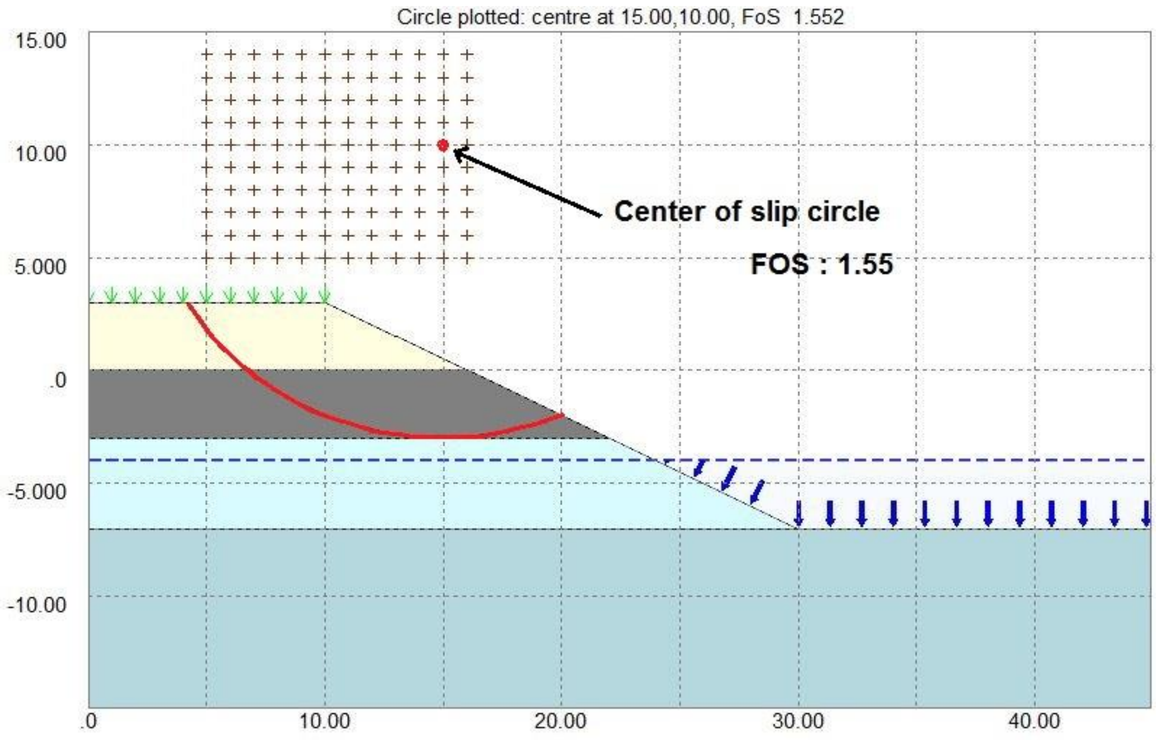


Figure 9. Slip circle with minimum factor safety of 1.55

Table 6. The factor of safety at various water levels with different shear strength parameters

Water level condition	Factor of Safety	
	Cu – Φ from reported	Cu – Φ from CIRIA correlation
Water level at FRL (+3 m)	2.2	2.5
Water level at DSL (- 4 m)	1.5	1.6
Water level at LTL (-6.5 m)	1.5	1.7

Slope stability analysis of approach and spill channels have to be revised once additional soil investigation data is available.

4 Slope protection works

The requirement of lining for the current purpose is to control seepage, increase hydraulic efficiency and increase resistance to erosion/ abrasion.

Rigid Lining materials are Stone-pitched lining, Burnt clay tile or brick lining, Precast cement concrete/stone slab lining, In-situ cement lime/concrete lining, Stone masonry lining, Soil cement lining, shotcrete lining and Asphaltic cement/concrete. Flexible Lining materials are Geo-membrane Like High-Density Polyethylene (HDPE), Polyvinyl Chloride (PVC), Low-Density Polyethylene (LDPE) Cover Comprising a Layer of Bentonite with Adequate/Earth/Burnt clay tile brick/Precast cement concrete, Bituminous or Bituminous/Asphaltic Felt Lining, Fibre reinforced plastic tissue as Phallic membrane, and Composite membrane/Rubber Lining.

As the construction of rigid lining is impossible underwater, the flexible lining is the only option. Based on cost and ease of execution, one of the following options can be adopted.

4.1 Geotextile mattresses

(a) Geotextile Grout Filled Mattresses (GGFM)

These are erosion-resistant concrete linings made from durable, permeable fabric forms that are filled with high-strength grout (figure 10). Fine-aggregate concrete is used in place of regular concrete because of its pumpability, strength, density and absorption resistance. The two layers of textile of a GGFM are connected via a vast array of internal restraining ties. Grouted in place, they adapt to uneven contours, curves and sub-grades as they are filled. GGFMs provide an economic and durable solution for erosion control, scour protection, shoreline protection and soil stabilization. They are highly effective in environmental and marine construction applications.

Figure 10. Geotextile grout filled mattress (Courtesy: web)

(b) Sand Filled and River Bank Protection Tubular Mattress

Two layers of engineered fabrics are stitched together at regular intervals and filled with sand. The bottom layer is composed of a woven geotextile and the top layer is composed of a special engineered composite geotextile that provides excellent abrasion resistance and durability. These are used as revetment along riverbanks, channel slopes and other waterway sections. Sand Filled and River Bank Protection Tubular Mattress is shown in Figures 11(a) and 11(b).

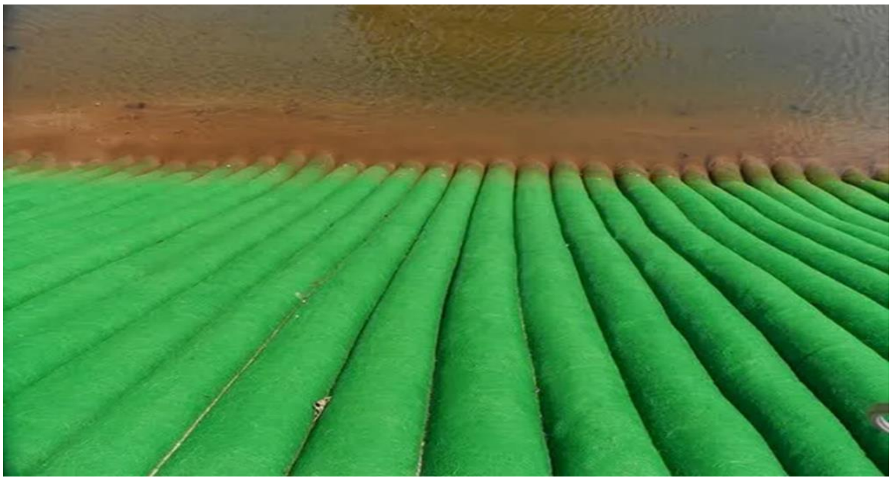


Figure. 11 (a)

Figure. 11(b)

Figure 11(a) and 11(b). Sand Filled and River Bank Protection Tubular Mattress

(Courtesy: web)

5 Pile foundation design

5.1 Pile foundations under piers

Pile foundation is proposed to support piers including a transportation corridor on piles. The pile capacities are calculated for various diameters and lengths, the same as given in **Table 7**. Cut off level of pile is 15 m below the existing ground level (- 12 m w.r.t MSL). Typical pile capacity calculations are given in enclosure 6. The proposed pile location under piers and transportation corridor is shown in figure12.

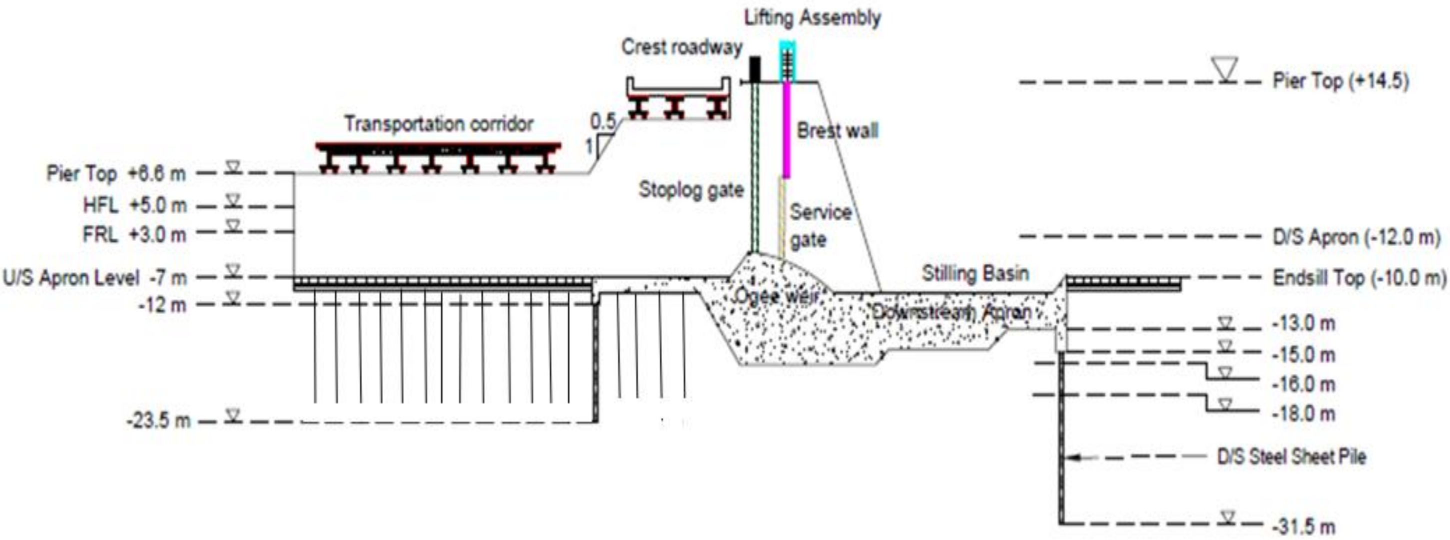


Figure 12. Pile location under piers and transportation corridor

Table 7. Pile capacities for different diameter with different length of piles

Diameter of the pile (m)	Length of the pile (m)	Cut-Off level of Pile	Safe skin friction (t)	Safe End bearing (t)	Safe Axial capacity (t)	Safe Uplift Capacity (t)
1	20	-12 m	130	50	180	130
	25		165	50	215	165
	30		200	50	250	200
	35		230	50	280	230
1.5	20		200	120	320	215
	25		245	120	365	270
	30		295	120	415	325
	35		345	120	465	380
2	20		260	210	470	310
	25		330	210	540	390
	30		395	210	605	470
	35		460	210	670	550
2.5	20		330	330	660	420
	25		410	330	740	525
	30		490	330	820	630
3	20		395	470	865	540
	25		490	470	960	675
	30		590	470	1060	810

The determined axial pile capacities are to be revised once the additional soil investigation data is made available.

6 Components of the spillway and its structural design:

6.1 Spillway crest

The spillway crest is the upper portion of the spillway over which water flows. 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 spillway crest is designed according to IS 13551-2019- Structural design of spillway pier and crest and IS 456-2000-Plain and reinforced concrete: Code of practice. Detailed design calculations are done in enclosure 7.

6.1.1 Loads considered

Weight of water over the crest, tailwater and horizontal water pressure are the major forces acting on the spillway crest.

6.1.2 Functions

The main function of the spillway crest is to allow a smooth flow of water over the spillway. The adoption of an ogee crest profile reduces the formation of negative pressures on the crest under the design head, thereby reducing the chances of cavitation.

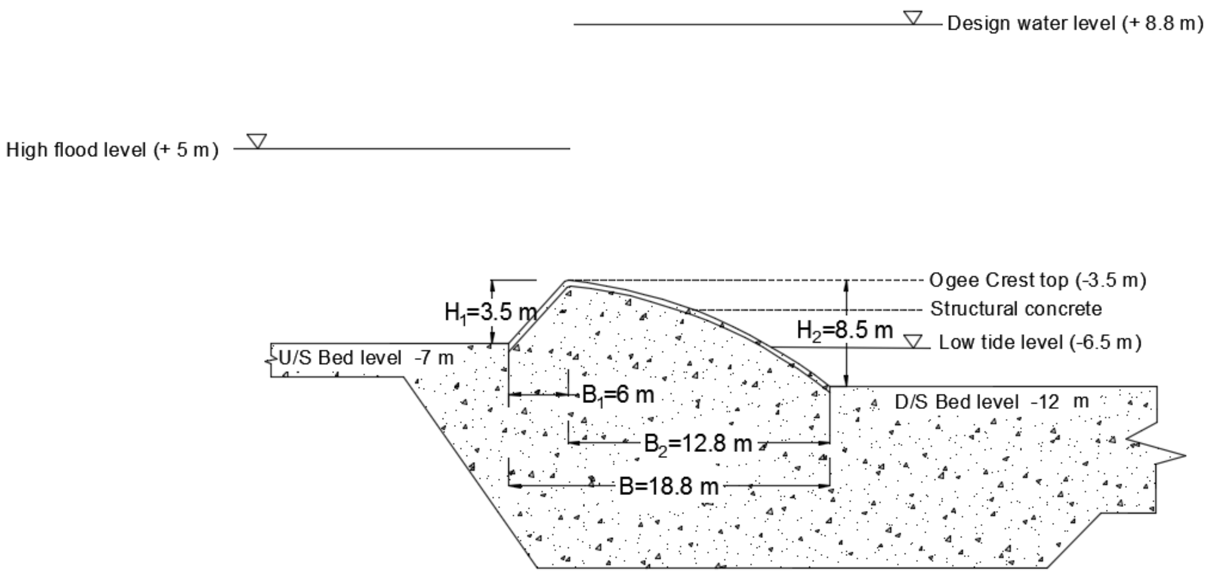


Figure 13. Ogee spillway

6.1.3 Levels and dimensions

Bottom width on upstream side, B_1	6 m
Height of crest on upstream side, H_1	3.5 m
Span of crest, B	18.9 m
Downstream side, B_2	12.8 m
Height of crest on downstream side, H_2	8.5 m
Crest level	- 3.5 m
Lowest bed level on downstream side	- 12 m
Lowest bed level on upstream side	- 7 m

6.1.4 Water level

High Flood Level, HFL (reservoir side)	+ 5 m
Design Water Level, DWL (sea side)	+ 8.8 m

6.1.5 Characteristics of reinforced concrete

Compressive strength of concrete, f_{ck}	35 N/mm^2
Grade of steel, f_y	Fe 500
Permissible tensile stress of steel, σ_{st}	275 N/mm^2
Permissible tensile stress in concrete in bending, σ_{cbc}	11.5 N/mm^2

6.1.6 Horizontal reinforcement

6.1.6.1 Reservoir side (upstream)

Thickness of ogee weir crest, d	2 m
Area of reinforcement (required), A_{st}	3451 mm^2
Area of reinforcement (provided), A_{st}	3927 mm^2
Provide 25mm dia bar @ 125mm c/c spacing	

6.1.6.2 Sea side (downstream)

Thickness of ogee weir crest, d	4 m
Area of reinforcement (required), A_{st}	2258 mm^2
Area of reinforcement (provided), A_{st}	2945 mm^2
Provide 25mm dia bar @ 160mm c/c spacing	

6.1.7 Vertical reinforcement

6.1.7.1 Reservoir side (upstream)

Thickness of ogee weir crest, d	2 m
Area of reinforcement (required), A_{st}	1148 mm^2
Area of reinforcement (provided), A_{st}	1885 mm^2
Provide 20mm dia bar @ 160mm c/c spacing	

6.1.7.2 Sea side (upstream)

Thickness of ogee weir crest, d	4 m
Area of reinforcement (required), A_{st}	5287 mm^2
Area of reinforcement (provided), A_{st}	6434 mm^2
Provide 32mm dia bar @ 125mm c/c spacing	

6.2 Breast wall

When the water level is higher than the high flood level, from the economical point of view of the height of gates, breast walls are provided above the high flood level up to the freeboard.

Breast wall is designed according to IS 11130- 1984- Criteria for Structural Design of Barrages and Weirs and IS 456-2000- Plain and reinforced concrete: Code of practice. Vertical breast wall is provided for vertical open gate. Detailed design calculations are done in enclosure 8.

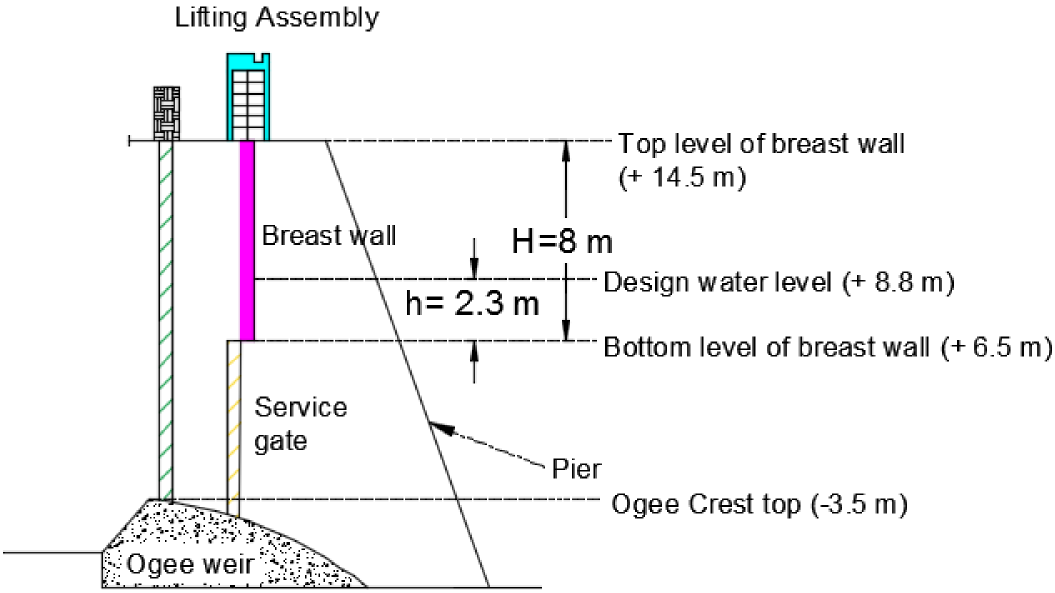


Figure 14. Breast wall

6.2.1 Loads considered

1.	Self-weight of the breast wall	2160 kNm/m
2.	Horizontal water pressure	19 kNm/m
3.	Hydrodynamic forces	6.6 kNm/m
4.	Seismic loads in X and -X direction	562 kNm/m

6.2.2 Load combinations

1. Water level is at high tide level
2. Water is at high tide level with seismic force
3. Water is at Design flood level
4. Water is at Design flood level with seismic force

6.2.3 Levels and Dimensions

Top-level of breast wall	+ 14.5 m
Bottom level of breast wall	+ 6.5 m
Height of breast wall, H	8 m
Height of water over the breast wall, h	2.3 m

6.2.4 Water level

High flood level, HFL (seaside)	+ 6.5 m
Design flood level in seaside, DFL (seaside)	+ 8.8 m

6.2.5 Characteristics of reinforced concrete

Compressive strength of concrete, f_{ck}	30 N/mm^2
Grade of steel, f_y	Fe 500
Permissible tensile stress of steel, σ_{st}	275 N/mm^2
Permissible stress in concrete in bending tension σ_{cbt}	2 N/mm^2
Permissible tensile stress in concrete in bending σ_{cbc}	10 N/mm^2

6.2.6 Reinforcement calculation

Thickness of the wall	600mm
Clear cover	50 mm
Minimum area of reinforcement, A_{stmin}, 0.24 % of gross area	1440 mm^2
Area of horizontal reinforcement (required), A_{st}	1275 mm^2
Area of horizontal reinforcement (provided), A_{st}	1885 mm^2
Provide 20mm dia bar @ 160mm c/c spacing	
Area of vertical reinforcement (required), A_{st}	290 mm^2
Area of vertical reinforcement (provided), A_{st}	1885 mm^2
Provide 20 mm dia bar @ 160 mm c/c spacing	

6.3 Baffle blocks and End sill

Baffle blocks and end sill are components of the stilling basin. When spillway gates are open, water rushes on the ogee crest which causes a hydraulic jump in the downstream. Hydraulic jump is caused due to kinetic energy which is generated during the flow of water. With the help of baffle blocks and end sill, kinetic energy will get dissipated which brings back the flow in downstream within a short distance and also prevent erosion of riverbed and banks. Stilling basin as suggested by U. S. Army Corps of Engineers (shown in the figure 15) is adopted. The main function of baffle blocks and end sill is to dissipate the energy and is designed according to IS 11527- 2013- Criteria for the structural design of energy dissipaters for spillways, IS 456-2000- Plain and reinforced concrete- Code of practice.

6.3.1 Data used for design

Froude number, F_1	3.6
Pre-jump depth, D_1	2.2 m
Post-jump depth, D_2	10.3 m
Floor level of stilling basin	- 12 m

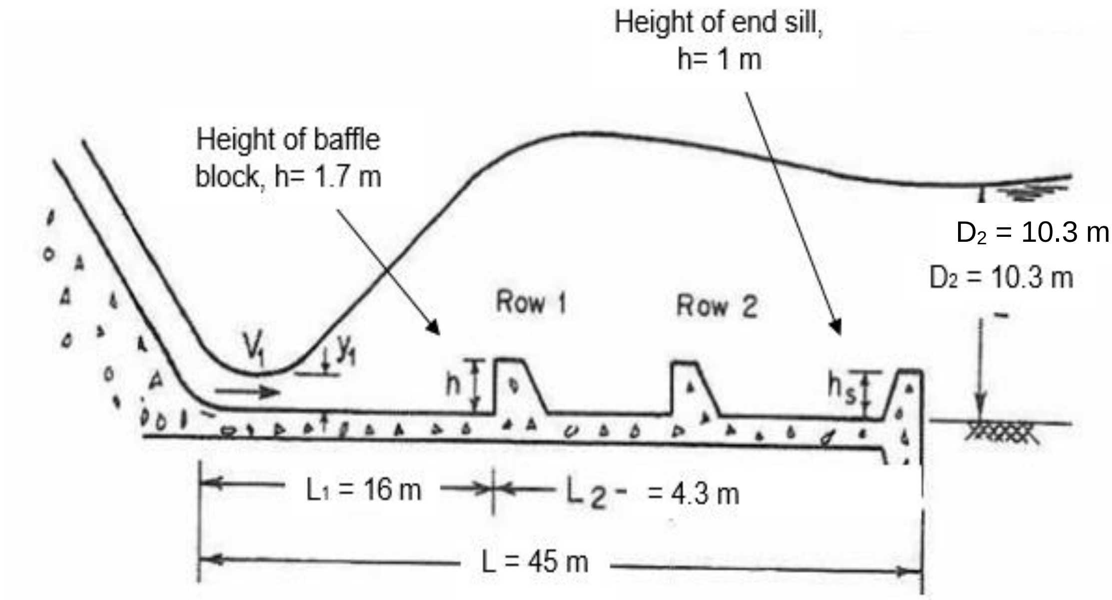


Figure 15. Proposed stilling basin

6.3.2 Characteristics of reinforced concrete

Permissible tensile stress of steel, σ_{st}	275 N/mm^2
Compressive strength of concrete, f_{ck}	35 N/mm^2
Grade of reinforcement, f_y	500 N/mm^2

6.3.3 Dimensions of baffle blocks and end sill

Distance from toe of ogee to face of row 1 baffle blocks, L_1	16 m
Height of baffle block in both row 1 and row 2, h	1.7 m
Width of baffle block	1.7 m
Spacing between baffle block	1.7 m
Distance from face of baffle block in row 1 to the face of baffle block in the second row, L_2	4.3 m
Height of end sill, h_s	1 m
Length of stilling basin, L	45 m

6.3.4 Reinforcement calculation

Based on the water levels, and function of baffle blocks and end sill, minimum reinforcement (i.e - 20 mm dia @ 300 mm center to center distance) is provided in both horizontal and vertical directions for baffle blocks and end sill.

7. References

i. CIRIA report R 143 1995, correlation between SPT N value and plasticity index (C_u/N)

ii. IS: 6403 (1981) – Code of practice for determination of bearing capacity of shallow foundations.

iii. IS 2911 part 1, section 2 – Code of practice for design and construction of pile foundation – Bored cast in-situ concrete piles

iv. IS 2314 (1986): Specification for steel sheet piling sections

v. IS 10084 (1982), part 1 – Criteria for design of diversion work for cofferdam

vi. IS 9527 (1981), part 4 – Code of practice for design and construction of port and harbor structure (Cellular sheet pile structure)

vii. IS 7894 - Code of practice for stability analysis of earth dams

viii. IS 10430 (2000): Criteria for design of lined canals and guidance for selection of type of lining.

ix. Soil mechanics and foundation engineering by Dr. K. Arora

x. IS 13551- 1992: Structural design of spillway pier and crest.

xi. IS 11527- 2013: Criteria for structural design of energy dissipaters for spillways

xii. IS 6512-2019: Criteria for design of solid gravity dams

xiii. IS 11130 -1984: Criteria for Structural Design of Barrages and Weirs

xiv. IS 12720- 2002: Criteria for structural design of spillway training walls and divide walls

xv. IS 4410- 1982-PART 9: Spillways and syphons

xvi. SP 55- 1993: Design Aid for Anchorages for Spillway Piers, Training Walls and Divide Walls

xvii. IS 6934- 1998: Hydraulic design of high ogee overflow spillway

xviii. IS 456-2000: Plain and reinforced concrete- Code of practice

ENCLOSURE 1

BORELOG- L1

CONTRACTOR	OWNER	COMPANY

BOREHOLE LOG

Project Name	**Marine Geo Technical Investigation Works for Kalpasar dam (Gulf of Khambhat) in GUJARAT, INDIA**	**BOREHOLE NO**	**L1**
Owner	**Kalpasar Department, Government of Gujarat**		
Client	**Gujarat State Petroleum Corporation LTD. (GSPC)**	**Proposed Coordinates**	**N: 21°48'25.82"** / **E: 72°36'8.09"** / **Datum** **WGS-84**
Location	**L-3 Alignment of Kalpasar Dam (Dahej Intertidal Region)**	**Positioned Coordinates**	**N: 21°48'25.76"** / **E: 72°36'8.09"** / **Zone** **43 N**
Project Code	**GFE-1-1219-06**	**Drilling Method**	**Rotary Drilling**
Termination Depth	**15.50 m below ground Level** / **EGL:(+) 2.9m m MSL** / **Ground Water Table: Submerged**	**Drilling Period**	**26 Jun to 29 Jun 2020**

BH Depth (m) Below Seabed		Drill Depth wrt MSL		Sample Type	Sample No	Standard Penetration Test (SPT)			SPT 'N' Value	Sample Recovery (%)	Soil Log	TCR (%)	SCR (%)	RQD (%)	Strata Description	Submerged Unit Weight (kN/m3)	Shear Strength (Kpa)			
From	To	From	To			15 cm	15 cm	15 cm									TV	PP	VST (Undisturbed)	VST (Remoulded)
0.00	0.50	2.90	2.40	UDS-1	1	--	--	--	--	84		--	--	--	Very Stiff, Very Dark Grayish Brown (10YR 3/2), Silty **CLAY (CH)**	--	123	123	--	--
0.5	0.95	2.40	1.95	SPT-1	2	3	5	9	14	95		--	--	--	Stiff, Very Dark Grayish Brown (10YR 3/2) Silty **CLAY(CH)**	--	--	--	--	--
1.50	2.00	1.40	0.90	UDS-2	4	--	--	--	--	76		--	--	--		--	98	98	--	--
3.0	3.45	-0.10	-0.55	SPT-2	5	1	1	3	3	96		--	--	--	Soft, Dark Grayish Brown (10YR 3/2) Silty **CLAY (CH)**	--	--	--	--	--
4.50	5.00	-1.60	-2.10	VST-1	6	--	--	--	--	--		--	--	--	Very Soft to Soft, Dark Grayish Brown (10YR 3/2) Silty **CLAY (CH)**	--	--	--	10	6
6.00	6.50	-3.10	-3.60	UDS-3	7	--	--	--	--	84		--	--	--	Very Stiff, Greenish Black (Glay1 10Y 2.5/1) Silty **CLAY (CH)**	--	159	172	--	--
7.50	7.95	-4.60	-5.05	SPT-3	8	5	9	17	26	92		--	--	--	(7.50 - 7.75m) Very Stiff, Dark Olive Gray (5Y 3/2) Silty **CLAY (CH)** (7.75 - 7.95m) Very stiff, Very Dark Gray (5Y 3/1) Silty **CLAY (CI)**	--	--	--	--	--
9.00	9.50	-6.10	-6.60	UDS-4	9	--	--	--	--	90		--	--	--	Very Stiff, Grayish Brown (10YR 5/2) Silty **CLAY (CI)**	--	110	106	--	--
10.50	10.95	-7.60	-8.05	SPT-4	10	6	11	15	26	95		--	--	--	Very Stiff, Dark Brown (10YR 3/3) Silty **CLAY (CI)**	--	--	--	--	--
12.00	12.50	-9.10	-9.60	UDS-5	11	--	--	--	--	84		--	--	--		--	147	135	--	--
13.50	13.95	-10.60	-11.05	SPT-5	12	7	15	24	39	90		--	--	--	Hard, Dark Brown (10YR 3/3) Silty **CLAY (CH)**	--	--	--	--	--
15.00	15.50	-12.10	-12.60	UDS-6	13	--	--	--	--	87		--	--	--	Very Stiff, Brown (10YR 4/3) Silty **CLAY (CI)**	--	147	147	--	--

Borehole L1 Terminated at 15.50m below Existing Ground Level (EGL).

Logged by	Ashish Rane	TCR	Total Core Recovery	MSL	Mean Sea Level	SPT	Standard Penetration Test	TV	Torvane
Checked by	Anil Kumbhare	SCR	Solid Core Recovery	CD	Chart Datum	DS	Disturbed Soil Sample	PP	Pocket Penetrometer
Drill Rig	Calyx Rotary Type	RQD	Rock Quality Designation	EGL	Existing Ground Level	UDS	UnDisturbed Soil Sample	VST	Vane Shear Test
Principle Soil Type	SAND		SILT		CLAY		ROCK		

Coastal Marine Construction & Engineering Ltd.

Lab at: C3 Cube, Mira-Bhayander Road, Mira Road (E), Thane- 401107

SUMMARY OF TEST RESULTS

Project	**Marine Geo Technical Investigation Works for Kalpasar dam (Gulf of Khambhat) in GUJARAT, INDIA**
Client	**Gujarat State Petrolium Corporation**
Project No.	**GFE-1-1219-06**
BH No.	**L1**
Date:	**09 October 2020**

SAMPLE DETAILS			On Board Lab Testing – Physical Properties						On Site Lab Testing – CLASSIFICATION & INDEX: Atterberg Limits							Grain Size Distribution					
Sample Type	Depth, m	Soil Classification	Water Content %	Bulk Density (kN/m3)	Submerged Unit Weight (kN/m3)	Dry Density (kN/m3)	Torvane Test (Kpa)	Pocket Penetrometer Test (Kpa)	Liquid Limit (**LL**) %	Plastic Limit (**PL**)%	Plasticity Index (**PI**) %	Shrinkage limit (%)	Free swell Index (%)	Specific Gravity	Void Ratio	Gravel %	Sand %	Silt %	Clay %	Fines % (Silt +Clay)	Relative density
UDS-1	0.00-0.50	CH	27.34	18.32	8.26	14.39	122.6	122.8	57.9	24.6	33.3	36.0	54.5	2.54	0.73	0	0	69	31	100	- -
SPT-1	0.50-0.95	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
UDS-2	1.50-2.00	CH	37.11	17.84	7.78	13.01	98.1	98.2	55.0	25.8	29.2	- -	- -	2.58	0.95	0	3	89	8	97	- -
SPT-2	3.00-3.45	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
UDS-3	6.00-6.50	CH	33.87	17.15	7.09	12.81	159.4	171.9	60.9	28.6	32.3	- -	- -	2.54	0.95	0	13	60	27	87	- -
SPT-3	7.50-7.75	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
	7.75-7.95	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	2.59	0.98	0	27	57	16	73	- -
UDS-4	9.00-9.50	CI	39.96	17.73	7.67	12.67	110.3	105.5	44.0	22.5	21.5	- -	- -	2.55	0.97	0	11	73	16	89	- -
SPT-4	10.50-10.95	CI	- -	- -	- -	- -	- -	- -	37.6	19.8	17.8	- -	- -	- -		- -	- -	- -	- -	- -	- -
UDS-5	12.00-12.50	CI	24.48	19.71	9.65	15.83	147.1	135.0	43.9	21.8	22.1	44.0	18.2	2.58	0.60	0	4	84	12	96	- -
SPT-5	13.50-13.95	CH	- -	- -	- -	- -	- -	- -	56.4	22.2	34.2	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
UDS-6	15.00-15.50	CI	35.40	17.84	7.78	13.18	147.1	147.3	44.0	22.8	21.2	- -	- -	2.59	0.93	0	16	72	12	84	- -
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- -	Bulk Sample	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
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SAMPLE DETAILS		On Shore Lab Testing – SHEAR STRENGTH TEST: Lateral & Vertical Load Tests				Consolidation							Proctor density test			Chemical tests					
Sample Type	Depth, m	Type of test	Cohession In Kpa	Angle In degrees	Soil UCS Kpa	Initial Void Ratio	Degree of Saturation %	Preconsolidation Pressure (kPa))	Compression Index	Swelling Pressure	Constant Head Permeability (cm/s)	Falling Head Permeability (cm/s)	Max. Dry Density (g/cc)	Optimum Moisture content (%)	Resonant column test	pH Value	Calcium Carbonate in %	Chloride Content in %	Sulphate Content in %	Organic matter content in %	Electrical Resistivity in ρ, Ω·cm
UDS-1	0.00-0.50	- -	- -	- -	80.37	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	8.04	1.25	1.34	0.16	4.30	466.10
SPT-1	0.50-0.95	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
UDS-2	1.50-2.00	TUU	56.40	7.40	- -	0.77	98	68.65	0.050	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
SPT-2	3.00-3.45	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
UDS-3	6.00-6.50	- -	- -	- -	69.73	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
SPT-3	7.50-7.75	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
	7.75-7.95	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	8.32	8.25	1.33	0.15	3.00	576.48
UDS-4	9.00-9.50	TUU	41.80	12.90	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
SPT-4	10.50-10.95	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
UDS-5	12.00-12.50	TUU	64.60	12.30	- -	0.41	97	112.78	0.012	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
SPT-5	13.50-13.95	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
UDS-6	15.00-15.50	TUU	70.90	12.00	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
- -		SPD	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	1.655	23.07	- -	- -	- -	- -	- -	- -	- -
- -	Bulk Sample	MPD	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	1.672	21.40	- -	- -	- -	- -	- -	- -	- -
- -		- -	- -	- -	- -	- -	- -	- -	- -	- -	$4.0X10^{-6}$	$10.21X10^{-6}$	- -	- -	- -	- -	- -	- -	- -	- -	- -
END OF REPORT																					
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Note:

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—Sample(s) was/were not drawn by laboratory.

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UDS= Undisturbed sample
SPT= Standard Penetration Test

NP= Non Plastic
CL= Inorganic clays of low to medium plasticity.
CI= Clay with Intermediate plasticit.
CH= Inorganic clays of high plasticity, fat clays.

TUU= Unconsolidated Undrained Triaxial Test
TCU=Consolidated Undrained Triaxial Test
TCD= Consolidated Drained Triaxial Test
DST= Direct Shear Test

SPD= Standard Proctor Density Test
MPD= Modified Proctor Density Test
CT= Cyclic Triaxial Test
UCS= Unconfined Compressive Strength Test

Coastal Marine Construction & Engineering Ltd. Geo Lab

Lab at: C3 Cube, Mira-Bhayander Road, Mira Road (E), Thane- 401107

TC-8445

TEST REPORT SUMMARY

Customer:	COMACOE, Mira Road			**Report no:**	**COMACOE/2020/041/01**	**ULR No**	**TC844520000000041F**
Project:	**Marine Geo Technical Investigation Works for Kalpasar dam (Gulf of Khambhat) in GUJARAT, INDIA**					Client :	**Gujarat State Petroleum Corporation Ltd. (GSPC)**
Sample Received on	14-07-2020	Sample Condition	OK	Test Started on	15-07-2020	Test Completed on **25-07-2020**	Report Dt: 10-10-2020
Sample Tested at	Lab	Lab Environment	Temp: 27 ± 2^{0} C			BH NO: **L1**	Project No: GFE-1-1219-06

SAMPLE DETAILS		CLASSIFICATION & INDEX															SHEAR STRENGTH TEST					Consolidation IS 2720 (Part 15), ASTM D2435					Chemical tests				
			Atterberg Limits				Physical Properties					Grain Size & Hydrometer Analysis IS 2720 (P-4), ASTM D421 & D422					Lateral & Vertical Loading Tests														
Sample Type	Sample Depth below seabed, m	Soil Classification IS 1498-1970, ASTM D2487-17	Liquid Limit in % IS 2720 (Part 5), ASTM D4318	Plastic Limit in % IS 2720 (Part 5), ASTM D4318	Plasticity Index PI %	Shrinkage limit in % IS 2720 (Part 6), ASTM D427	Water (Moisture) content in % (IS 2720 (Part-2), ASTM D2216)	* Bulk Density (kN/m^3)	* Submerged Unit Weight (kN/m^3)	* Dry Density (kN/m^3)	Specific Gravity IS 2720 (P-3, Sec.1 & 2), ASTM D854	Gravel %	Sand %	Silt %	Clay %	Fines % (Silt +Clay)	Triaxial Shear Test -Cohesion- in Kpa (UU) IS 2720 (Part 11), ASTM-D2850	Triaxial Shear Test Angle- in degrees (UU) IS 2720 (Part 11), ASTM-D2850	Direct Shear test- Cohesion- in Kpa IS 2720 (Part 13), ASTM D3080	Direct Shear test- Angle- in degrees IS 2720 (Part 13), ASTM D3080	UCS- Soil in Kpa IS 2720 (Part 10), ASTM D2166	Initial Void Ratio	Degree of Saturation %	Preconsolidation Pressure (kPa))	Compression Index	Free Swell Index in % IS 2720 (Part 40), ASTM D4546	pH Value (IS 2720 P26/ASTM D4972)	Calcium Carbonate in % (IS 2720 (P-23) , ASTM D4373)	Chloride Content in % (IS 3025 -P32)	Sulphate in % IS 2720 (P 27), ASTM C1580)	Organic matter content in % ASTM D2974
UDS-1	0.00-0.50	CH	58.0	25.0	33.0	36.0	27.34	18.32	8.26	14.39	2.54	0	0	69	31	100	- -	- -	- -	- -	80.37	- -	- -	- -	- -	54.50	8.04	1.25	1.34	0.16	4.30
SPT-1	0.50-0.95	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
UDS-2	1.50-2.00	CH	55.0	26.0	29.0	- -	37.11	17.84	7.78	13.01	2.58	0	9	89	8	97	56.40	7.4	- -	- -	- -	0.765	98.0	68.65	0.050	- -	- -	- -	- -	- -	- -
SPT-2	3.00-3.45	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
UDS-3	6.00-6.45	CH	61.0	29.0	32.0	- -	33.87	17.15	7.09	12.81	2.54	0	13	60	27	87	- -	- -	- -	- -	69.73	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
SPT-3	7.50-7.75	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
	7.75-7.95	- -	- -	- -	- -	- -	- -	- -	- -	- -	2.59	0	27	57	16	73	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	8.32	8.25	1.33	0.15	3.00
UDS-4	9.00-9.50	CI	44.0	23.0	22.0	- -	39.96	17.73	7.67	12.67	2.55	0	11	73	16	89	41.80	12.9	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
SPT-4	10.50-10.95	CI	37.6	19.8	17.8	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
UDS-5	12.00-12.50	CI	44.0	22.0	22.0	44.0	24.48	19.71	9.65	15.83	2.58	0	4	84	12	96	64.60	12.3	- -	- -	- -	0.412	97.0	112.78	0.012	18.20	- -	- -	- -	- -	- -
SPT-5	13.50-13.95	CH	56.4	22.2	34.2	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
UDS-6	15.00-15.50	CI	44.0	23.0	21.0	- -	35.40	17.84	7.78	13.18	2.59	0	16	72	12	84	70.90	12.0	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
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-- * This Marked is Not in NABL scope.

NP= Non Plastic

CL= Inorganic clays of low to medium plasticity.

CI= Clay with Intermediate plasticit.

CH= Inorganic clays of high plasticity, fat clays.

IS= Insufficient Sample

Failure Type	
A	Bulge
B	Inclined Shear
C	Multiple Shear
D	Vertical Shear

Reviewed by

Authorized by
(Name & Designation)

ENCLOSURE 2

CIRIA CORRELATION BETWEEN COHESION AND SPT N FOR COHESIVE LAYERS

6.2 THE SPT IN COHESIVE SOIL

From the earliest use (for example, Terzaghi and Peck, 1948) it has been appreciated that penetration resistance in cohesive soil is broadly a function of undrained shear strength c_u. The precise relationship between undrained shear strength and SPT N is controlled, however, by a number of factors. These include plasticity, sensitivity and fissuring. Equipment factors, such as those described in Section 5, have also undoubtedly contributed to the wide range of the ratio c_u/N reported in the literature. And it is well recognised that undrained shear strength is not an unique soil parameter, but depends strongly upon the method by which it is determined (for example, see Wroth, 1984), and the precise orientation of any planes of weakness (i.e. fissures) in the test specimen.

For overconsolidated clays, Stroud (1974) has reported good correlations between N and c_u. (Figure 31). The strength of these correlations results from the standardisation of the SPT in the UK, the relatively small influence of British drilling methods on these types of ground, and the fact that the undrained shear strengths were determined in a single way, using triaxial compression tests on 102mm diameter specimens. It should also be noted that the relationships were developed from comparison of depth profiles of both c_u and N on given sites, rather than comparison of individual results. These depth profiles show that N values are less scattered than undrained shear strength values, and this suggests that the Standard Penetration Test may be a powerful technique for determining undrained strength.

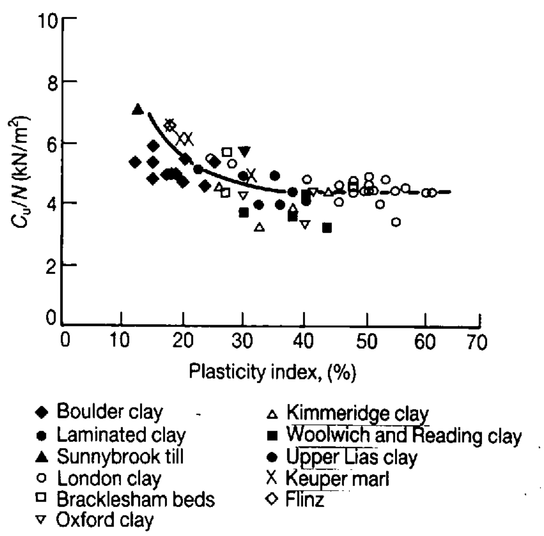

Figure 31 *Correlation between N value and undrained shear strength, c_u for insensitive clays (after Stroud, 1974)*

Some comparisons between penetration resistance and undrained shear strength have given a very wide range of c_u/N values. For example, de Mello (1971) shows values with c_u/N ratios apparently varying between 0.4 and 20. The contrast between the very tight grouping of results quoted by Stroud (1974) and the wide range of values given by de Mello's data is explained because:

1. Stroud's data relates only to insensitive overconsolidated clays, whereas some of de Mello's data undoubtedly is derived from soft sensitive clays.

ENCLOSURE 3

DESIGN CALCULATIONS OF COFFERDAM

ACTIVE AND PASSIVE EARTH PRESSURE CALCULATIONS

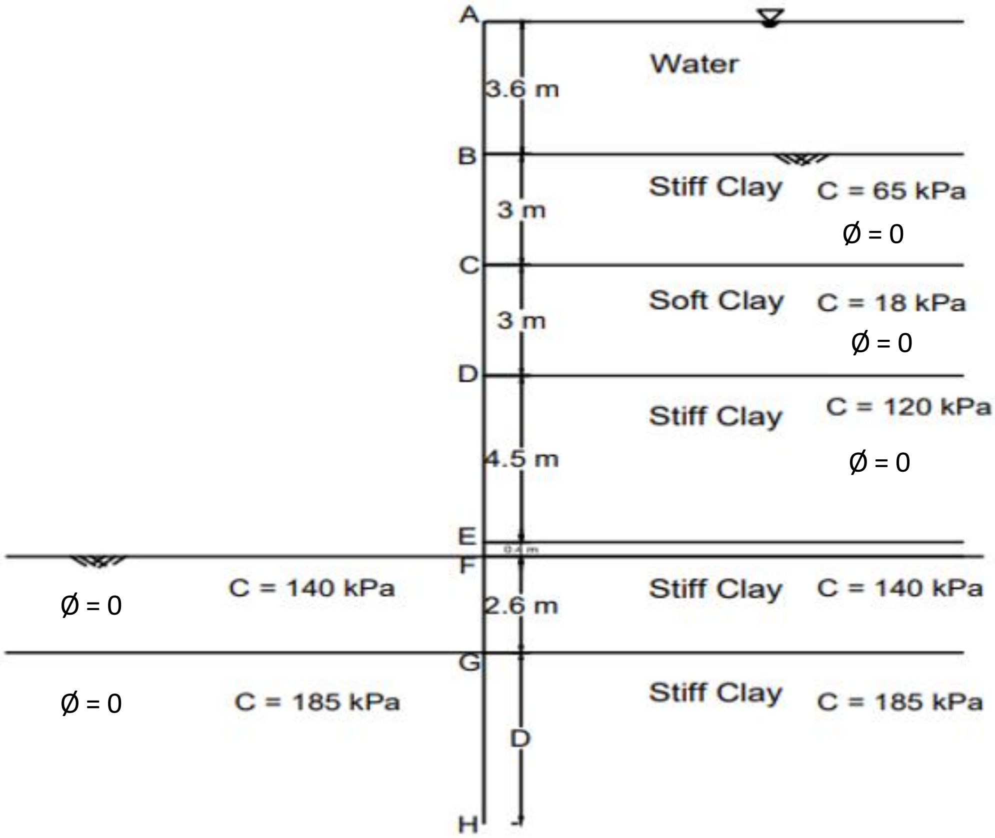

Figure1 : Soil profile

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 pressure 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,

(just above)

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

$P_a = 3.6 \times 9.81 =$ *35.32 kPa*

(just below)

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

$P_a = (3.6 \times 9.81)-(2 \times 65 \times \sqrt{1})=$ *-94.68kPa*

At C,

(just above)

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

$P_a = (6.6 \times 9.81)+(1 \times 8.02 \times 3)-(2 \times 65 \times \sqrt{1}) =$ *-41.19kPa*

(just below)

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

$P_a = (6.6 \times 9.8)+(1 \times 8.02 \times 3)-(2 \times 18 \times \sqrt{1})=$ *52.81 kPa*

At D,

(just above)

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

$P_a = (9.6 \times 9.81)+(1 \times (8.02 \times 3)+(7.1 \times 3))-(2 \times 18 \times \sqrt{1}) =$ *103.54 kPa*

(just below)

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

$P_a = (9.6 \times 9.81)+(1 \times ((8.02 \times 3)+(7.1 \times 3))-(2 \times 120) =$ *-100.46kPa*

At E,

(just above)

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

$P_a = (14.1 \times 9.81)+(1 \times ((8.02 \times 3)+(7.1 \times 3)+(7.38 \times 4.5))-(2 \times 120 \times \sqrt{1}))$

$P_a = -23.11kPa$

(just below)

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

$P_a = (14.1 \times 9.81)+(1 \times ((8.02 \times 3)+(7.1 \times 3)+(7.38 \times 4.5))-(2 \times 140 \times \sqrt{1})$

$P_a = -63.11kPa$

At F,

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

$P_a = (14.5 \times 9.81)+((18.02 \times 3)+(7.1 \times 3)+(7.38 \times 4.5)+(0.4 \times 3.72)) -$

$(2 \times 140 \times \sqrt{1})=-55.697kPa$

At G,

(just above)

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

$P_a = (17.1 \times 9.81)+(((8.02 \times 3)+(7.1 \times 3)+(7.38 \times 4.1)+(3 \times 8.72))-(2 \times 140 \times \sqrt{1})$

$P_a = -7.52kPa$

(just below)

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

$P_a = (17.1 \times 9.81)+(1 \times ((8.02 \times 3)+(7.1 \times 3)+(7.33 \times 4.5)+(3 \times 8.72)) -$

$(2 \times 18.5 \times \sqrt{1}) = -97.52kPa$

At H,

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

$P_a = ((17.1+D)9.81)+(((8.02 \times 3)+(7.1 \times 3)+(7.38 \times 4.5)+(3 \times 8.72)+(8.72 \times D))$

$-(2 \times 18.5) = 18.53D-97.52$

Passive earth pressure (P_P)

Passive earth pressure at different points are calculated below,

At F,

P_p = 2 x 140 x √1= *280 kPa*

At G

(just above)

P_p = (1 x 2.6 x 8.72)+(9.81 x 2.6)+(2 x 140 x √1)= *328.18 kPa*

(just below)

P_p = (1 x 2.6 x 8.72)+(9.81 x 2.6)+(2 x 85 x √1)= *418.18 kPa*

At H,

P_p = (1 x (2.6+D)8.72)+(9.81 x (2.6+D))+(2 x 185 x √1) = *418.18+18.53D*

Active and Passive Moment Calculations

Active moment

From active earth pressure diagram, moment about point H is

$$M_a = (\frac{1}{2} \text{ x } (18.53D-97.52)(D - 5.26)\frac{(D-5.26)}{3})+(52.81 \text{ x } 3 \text{ x } (D+7.5+\frac{3}{2}))+$$

$$(\frac{1}{2} \text{ x } 50.69 \text{ x } 3(D+7.5+\frac{3}{3}))+(\frac{1}{2} \text{ x } 3.6 \text{ x } 35.32(D+13.5+\frac{3.6}{3}))$$

$$M_a = (3.09D-16.25)(D^2+27.6710.52D))+158.43D+1425.87$$

$$+76.035D+646.3+63.576D+934.57$$

$$M_a = 3.09D^3+85.5D-32.51D^2-16.25D^2-449.64+170.95D+298.041D+3006.74$$

$$M_a = 3.09D^3-48.76D^2+554.49D+3456.38$$

Passive moment

From passive earth pressure diagram, moment about point H is

$$M_p = (418.18 \text{ x D x } \frac{D}{2})+(\frac{1}{2} \text{ x } 18.53D \text{ x D x } \frac{D}{3})+(2 \text{ x } 2.6 \text{ x } (D+\frac{2.6}{2}))+$$

$$(\frac{1}{2} \text{ x } 48.18 \text{ x } 2.6 \text{ x } (D+\frac{2.6}{3}))$$

$$M_p = 209.1\ D^2 + 3.09\ D^3 + 728\ D + 946.4 + 62.634\ D + 54.283$$

$M_p = 3.09\ D^3 + 209.1\ D^2 + 790.63\ D + 1000.68$

Taking a factor of safety of 2 on passive side,

$\frac{M_p}{2} = 1.544D^3 + 104.55D^2 + 395.315D + 500.34$

Equating the moments on active and passive side,

$3.09D^3 - 48.76D^2 + 554.49D + 3456.38 = 1.544D^3 + 104.55D^2 + 395.315D + 500.34$

$1.544D^3 - 153.31D^2 + 159.175D + 2956.04 = 0$

Depth of penetration in clay layer = 5.088m

Therefore, Total Depth of penetration of Coffer dam = 5.088+2.6

$$D = 7.7\ m$$

Total Height of Coffer Dam (H):

H = Free Board + D + Height of soil and water to be retained

Considering the free board as 1.5 times the height of wave (according to IS: 10084 (part 1) -1982), free board = 1.5 x h_w

From CWPRS report, height of wave = 5.3 m

Free board = 1.5 x 5.3 = 8 m

$$H = 8 + 7.7 + 14.5$$

H = 30.5 m

ENCLOSURE 4

CHECK FOR STABILITY OF COFFERDAM AGAINST VARIOUS PARAMETERS

CHECK FOR STABILITY

Factor of safety

Height of Coffer dam, H = 30.5 m

Taking diameter, D = 10.19m

From *IS:9527 (part 4)*,

Effective Width of cell, B = 8.51 m

1) **Cell shear**

Vertical shear force(v) = $\frac{1.5M}{B}$

M = (3.09 x 5.088^3)-(48.76 x 5.088^2)+(554.49 x 5.088)+3456.38

M = 5422.34 kN.m

v = $\frac{1.5*5422.34}{8.51}$

v = 955.76 kN

Soil shear strength, s= $\frac{1}{2}\gamma kH^2(\tan\phi + f)$

H – Height of soil above ground level

f – friction co-efficient = 0.3

s = $\frac{1}{2}$ x 15 x 0.6 x 22.5^2 (tan 30+0.3)

s = 1998.713 kN

FOS = $\frac{s}{v}$ = $\frac{1998.713}{955.76}$

FOS =2.09

2) **Sliding**

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

FOS = $\frac{\mu W + Pp}{Pa}$

Taking μ= tan ϕ = tan 30

P_p = $(\frac{280+328.18}{2}$ x 2.6) + $(\frac{418.18+512.46}{2}$ x 5.088)

P_p = 3158.182 kN/unit width

P_a = $(\frac{1}{2}$* x 3.6 x 35.32) + $(\frac{52.8+103.5}{2}$ x 3)

P_a = 298.04 kN/unit width

W = $(\frac{10+20}{2})$ x 30.5 x 8.51

W = 3893.32 kN/unit width

FOS =$\frac{((\tan 30)(4212.45))+3132.67}{301.033}$

FOS =18.485

3) Tilting/Overturning

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

FOS = $\frac{Mp}{Ma}$

FOS= *2*

Hence OK

B $\geq\sqrt{\frac{6P_aZ}{\gamma_a H}}$

Z – point of action of P_a from base

From active earth pressure diagram,

Z = $\frac{\left(52.81*3*\left(12.588+\frac{3}{2}\right)\right)+\left(\frac{1}{2}*50.69*3*\left(12.588+\frac{3}{3}\right)\right)+(\frac{1}{2}*3.6*35.32*\left(18.588+\frac{3.6}{3}\right))}{298.04}$

Z = $\frac{4523.167}{298.04}$

Z = 15.176m(from base)

B $\geq\sqrt{\frac{6*298.04*15.176}{15*30.5}}$

B $\geq$ 7.701

8.51 $\geq$ *7.701*

4) Bursting of cell

Interlock tension =T_{all} =150 t/m (from IS:9527(part iv))

FOS = $\frac{T_{all}}{T_{max}}$

P = $(k\gamma_a H' + \gamma_w H_w)$

H' – depth of soil upto that level

H' = 10.9m, k = 0.6

P = (0.6 x 15 x 10.9)+(9.81 x 3.6)

P = 133.416 KP_a

T_{max} = $\frac{PD}{2}$ = $\frac{133.416*10.19}{2}$

T_{max} = 679.754 kN/m

FOS = $\frac{T_{all}}{T_{max}}$

FOS = $\frac{1500}{679.754}$

FOS = 2.2

5) For coffer dam embedded in clay

FOS = $\frac{Q_{ult}}{fill\ \ load}$

FOS = $\frac{Q_{ult}}{\gamma_a\, BH}$

For clays, Q_{ult} = 5.7 CB

Average Cohesion, C = $\frac{(2.6*140)+(5.0881*185)}{(2.6+5.088)}$

C = 169.78 kPa

Q_{ult} = 5.7 x 169.78 x 8.51

Q_{ult} = 8235.59 kN/m^2

FOS = $\frac{8235.59}{15*8.51*30.5}$

FOS = 1.762

ENCLOSURE 5

SLOPE STABILITY ANALYSIS OF APPROACH AND SPILL CHANNELS

Slope stability analysis of approach and spill channels for 1 V: 2 H slope based on reportedand shear strength parameters obtained from CIRIA correlation is carried out using Oasys Slope 19.0.

The analysis is carried out for the followingwater levels conditions.

i. Water level at +3 m (FRL) w.r.t MSL of approach channel
ii. Water level at - 4 m (DSL) w.r.t MSL of approach channel
iii. Water level at -6.5 m (LTL) w.r.t MSLof spill channel

The analysis is carried out for various slopes and the slope yielding minimum factor of safety is 1 V : 2 H. The schematic diagram of recommended slope of approach and spills channel with interpretedand reported and shear strength parameters is shown in figures 1 and 5.

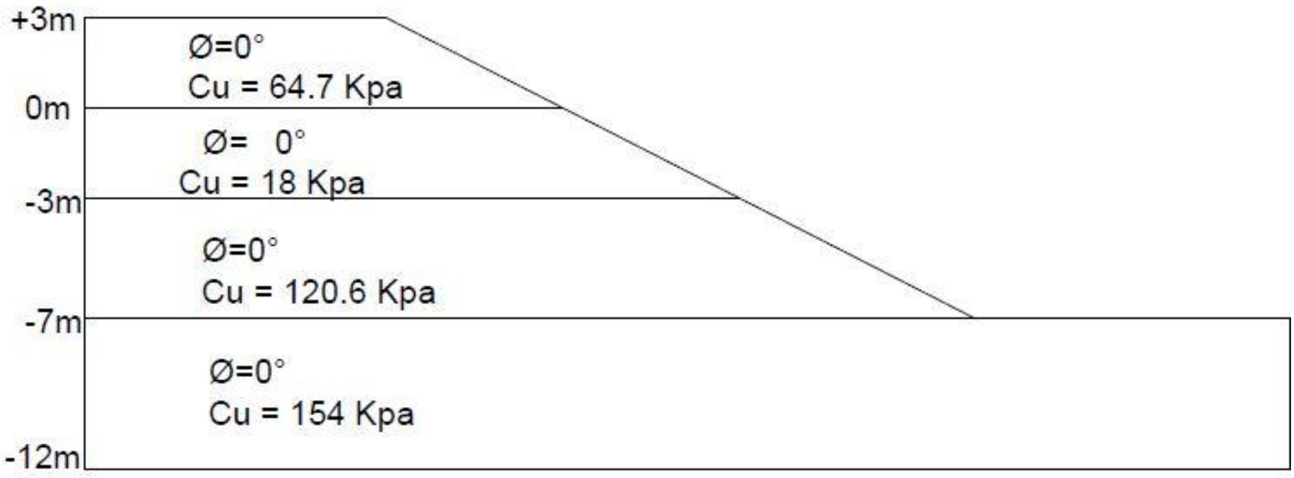

Figure 1.Schematic diagram of approach and spill channelfor slope 1 V : 2 H

The slip circle of approach and spill channels at various water level conditions considering shear strength parameters obtained from CIRIA correlation are shown in Figure 2,3 and 4 as shown below. Similarly figures 6, 7 and 8 shows slip circles of approach and spill channels considering reported shear strength parameters.

Slip circle considering water level at + 3 m (FRL) of approach channel

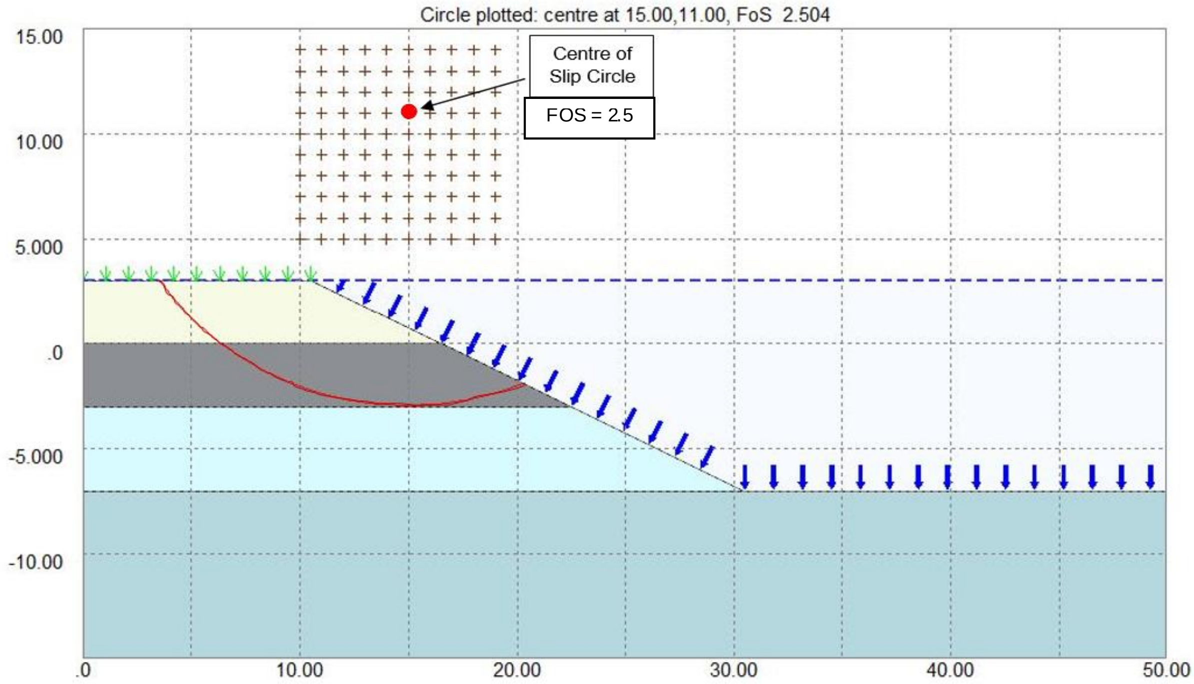

Figure 2. Slip circle with minimum factor safety 2.5

Slip circleconsidering water level at – 4 m (DSL) of approach channel

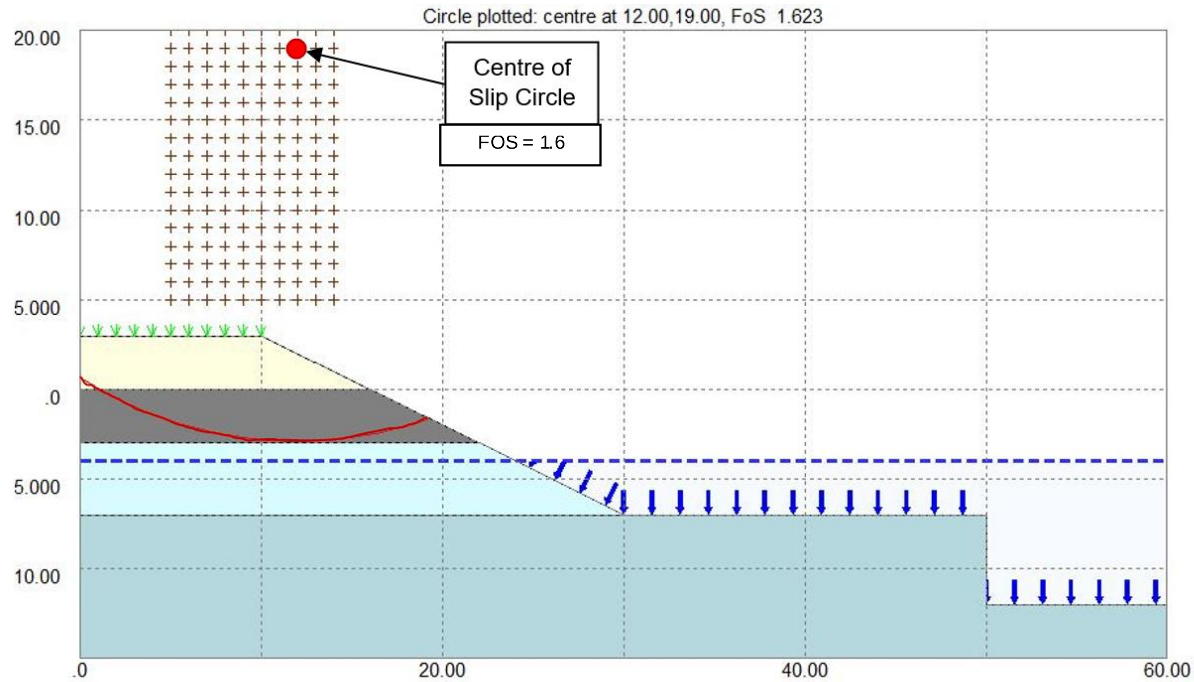

Figure 3. Slip circle with minimum factor safety 1.6

Slip circle considering water level at – 6.5 m (LTL) of spill channel

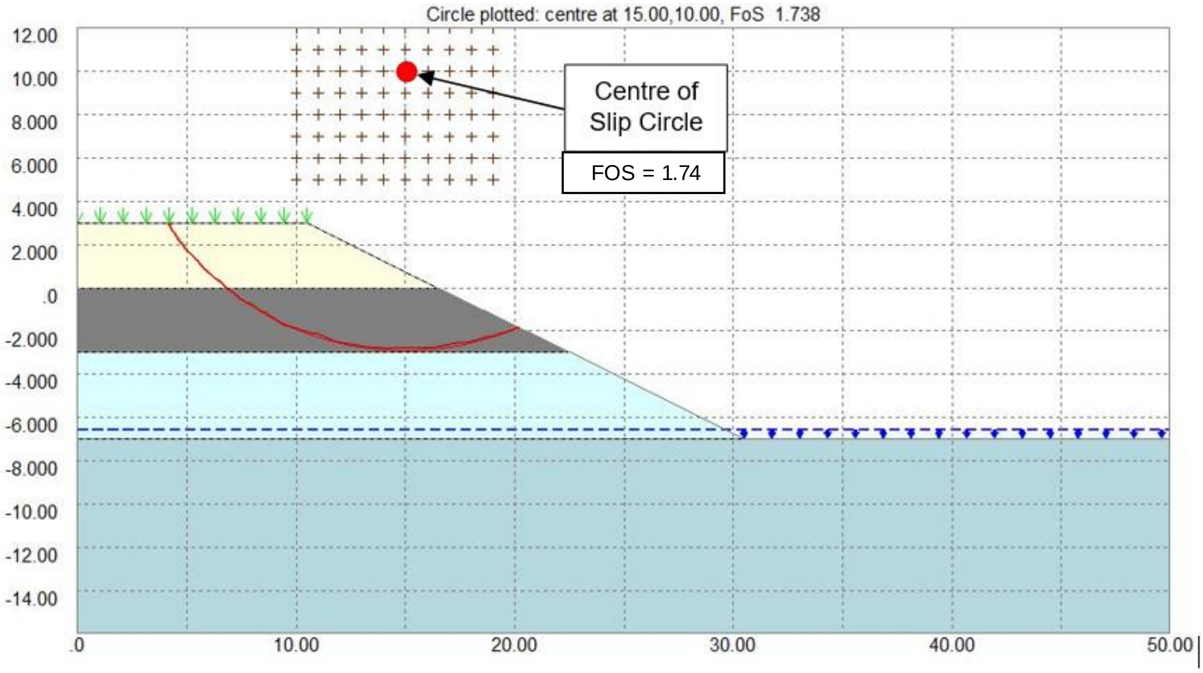

Figure 4. Slip circle with minimum factor safety 1.7

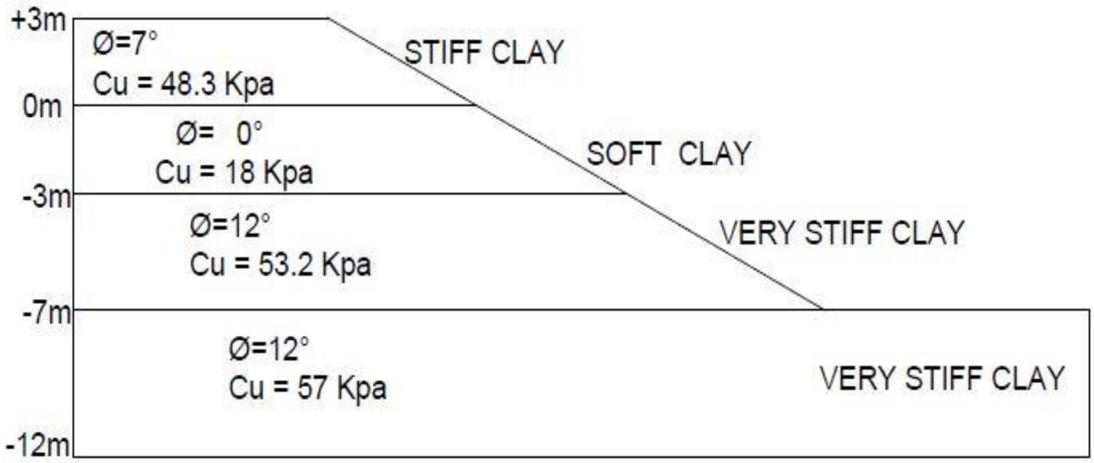

Figure 5. Cross section of approach channel model for slope 1 V : 2 H

Graphical output considering water level at + 3 m (FRL) of approach channel

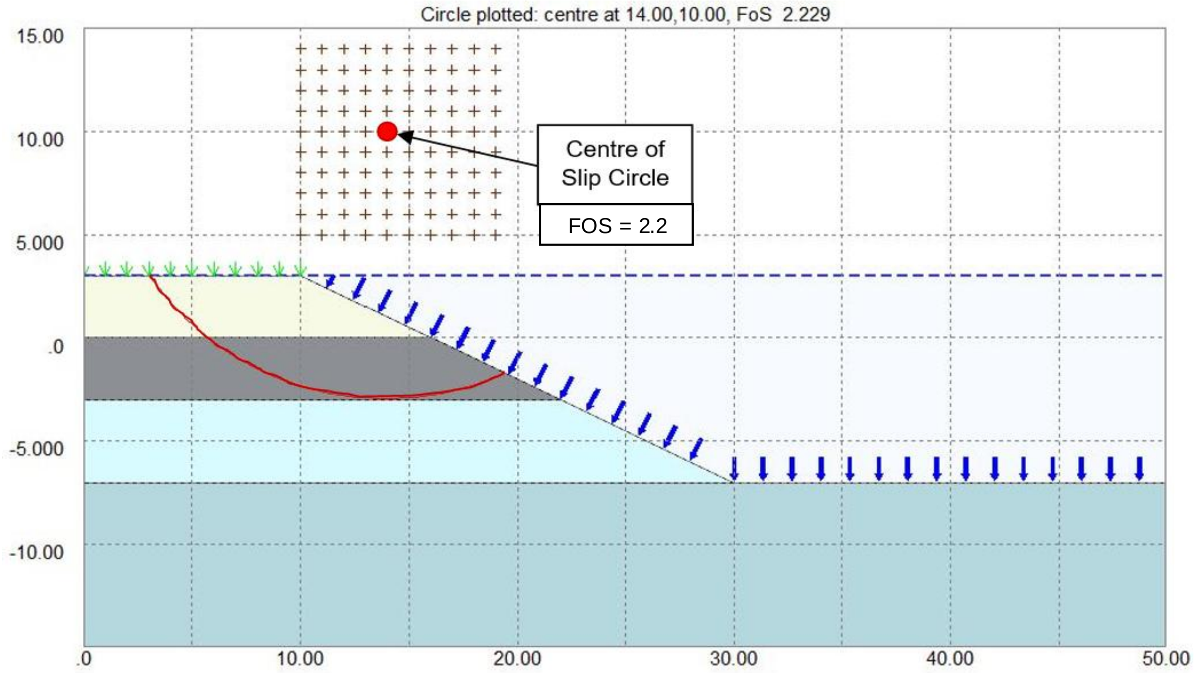

Figure 6. Slip circle with minimum factor safety 2.23

Graphical output considering water level at – 4 m (DSL) of approach channel

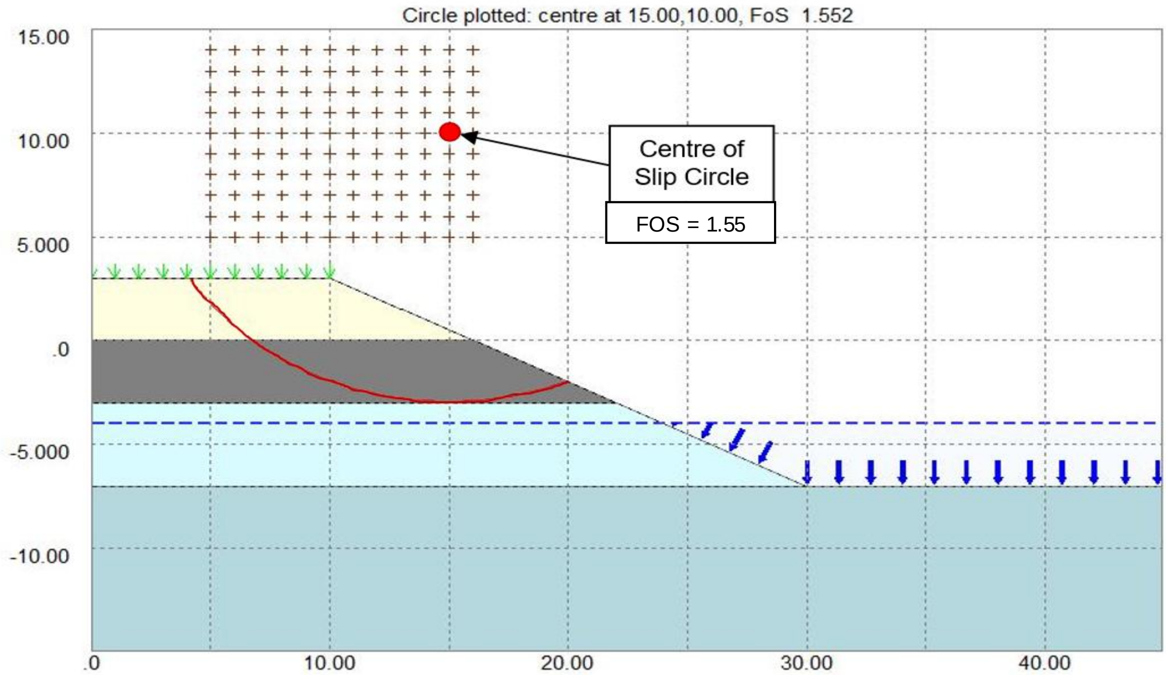

Figure 7. Slip circle with minimum factor safety 1.55

Graphical output considering water level at – 6.5 m (LTL) of spill channel

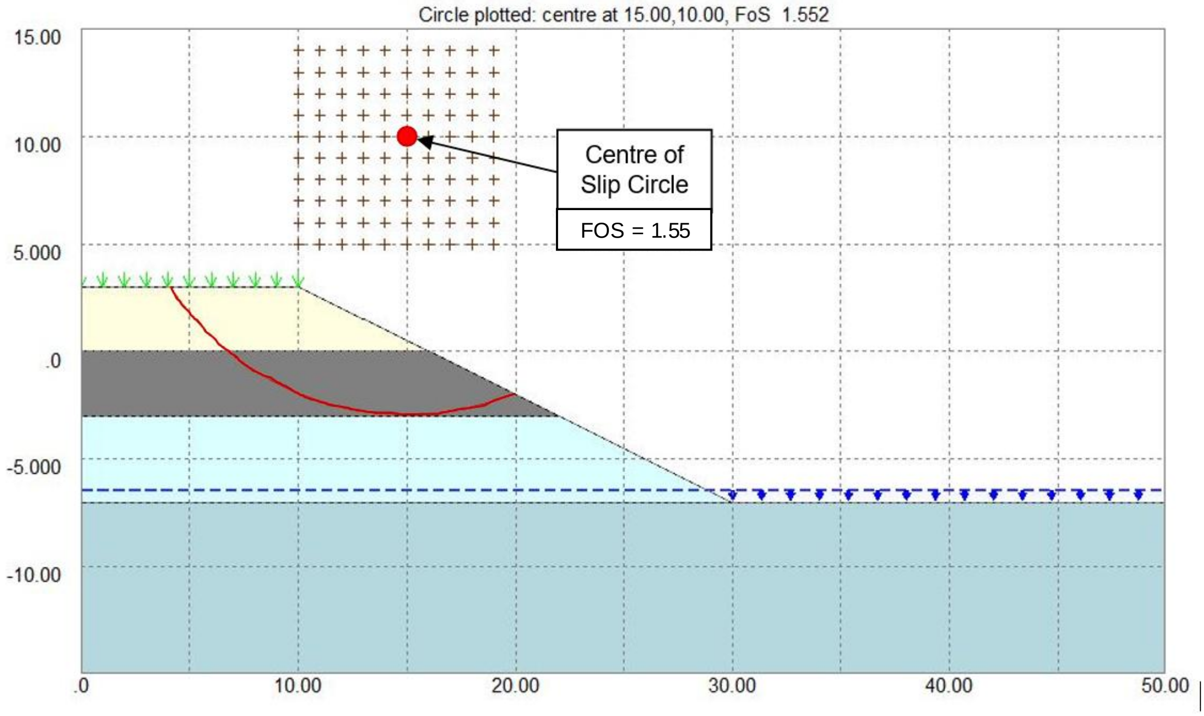

Figure 8. Slip circle with minimum factor safety 1.55

ENCLOSURE 6

PILE CAPACITY CALCULATIONS

Project: Kalpasar dyke - Spillway

Estimation of Axial and Uplift capacities of piles (As per IS : 2911 (Part 1 / Sec 2))

Assumption:

Subject: Safe Axial and Uplift Capacities of Pile

1) Critical depth is considered as 15 x diameter of pile

Input:

Type of pile:	**Bored Cast - insitu concrete piles**	
Diameter of pile	3	m
Cut-off Level of pile	15	m below Existing Ground Level
Length of the pile, below cut-off level	30	m
Termination level of the pile	**45**	m
Depth of GWT, below ground level	0	m
Depth of GWT, below cut off level	0	m
Bore Hole No	**BH L1**	
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, Nγ	0.00	(from IS:6403, table -1) Φ 0 For End bearing
Bearing capacity factor, Nq	0	(from IS:2911-part-1-sec-2-Fig.1)
Area of pile at tip, Ap	7.065	m^2
Adhesion Factor (α)	0.28	(from IS:2911-part-1-sec-2-Appendix B 2.1, Note 1)
Bearing Capacity Factor, Nc	9	

Soil strata :

Layer	Depth (m)	Thickness (m)	Soil Type	Phi (deg)	Cohesion (kN/m^2)	Adhesion Factor (α)	Earth Pressure Coefficient (K)
Layer-I	15-45	30.0	Very stiff silty clay	Φ= 0	C = 185.6	0.28	1

Formulae:

Pile Capacity **Qu** $= A_p\,[(0.5 \times D \times \gamma \times N_\gamma) + (P_d \times N_q)+(N_c \times C_p)] + [(\Sigma^{D}_{i=1} K \times P_{di} \times \tan\delta \times A_{si})+(\alpha C' \times A_s)]$

Critical depth D_c = 15 x dia of pile **15** **x** **3.00** 45 m

Effective Overburden Pressure at Critical Depth = 450 kN/m^2

Effective Overburden Pressure at tip of the pile = 300

	Depth below ground level / Cut-off level	Depth (m)	Linear Pressure variation		Pressure variation after considering critical depth	
Pressure calculations:	15 m / 0 m	0	0	kN/m^2	0.0	kN/m^2
	40 m / 30 m	30.0	300	kN/m^2	300.0	kN/m^2

<u>Calculation of Friction component:</u>

layer- I **Very stiff silty clay**

$Q_{s1} = [(\Sigma^{D}_{i=1} K \times P_{di} \times \tan \delta \times A_{si}) + (\alpha C' \times A_s)]$

Qs1 = 1x[(0+300)/2 x tan0xπx 3x 30] + (0.28x185.6xπx x3x30) **=** 14693.6 kN

Skin friction due to soil = Qs = sum (Qs1) = 14693.6 kN

= 1469.4 tons

Safe skin friction due to soil = 588 tons **(Factor of safety =2.5)**

<u>Calculation of End bearing Component:</u>

$Q_e = A_p [(0.5 \times D \times \gamma \times N_\gamma) + (P_d \times N_q) + (N_c \times C_p)]$

Qe =7.065 x [(0.5 x3 x10 x 0) + (300x0)+(9x185.6)] = **11801.4** **kN**

Ultimate End bearing component of pile = 1180 tons

Safe end bearing component of pile = **472** **tons** **(Factor of safety =2.5)**

Safe Axial capacity of pile = **588 + 472** **tons**

= 1060 **tons**

Uplift Pile Capacities:

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

Weight of pile (W) **=** 3179.25 kN

Total Skin friction = Qs **=** 14693.6 kN

Ultimate uplift capacity of pile Qu = Qs + W **=** (14693.6 / 3) + 3179.25 kN **(Factor of safety =3)**

= 8077 kN

Safe uplift capacity of pile = 808 tons

ENCLOSURE 7

STRUCTURAL DESIGN OF OGEE WEIR CREST

Ogee weir crest is designed as per IS 13551- 1992-Structural design of spillway pier and crest and IS 456-2000- Plain and reinforced concrete: Code of practice. Thickness of ogee weir crest for reservoir side is 2m and for sea side it is 4m.

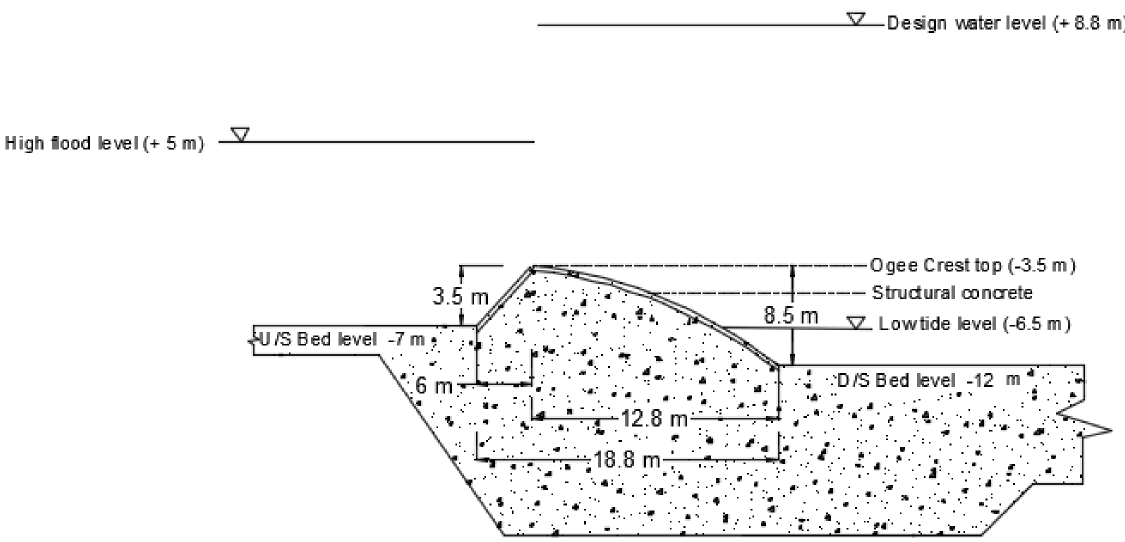

Fig 1 Ogee spillway

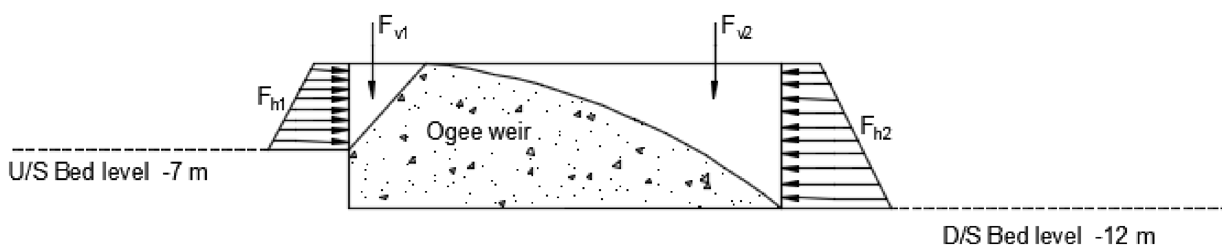

Fig 2 Force diagram of ogee weir crest

1.1 <u>Data used for the design</u>

Compressive strength of concrete	$35 N/mm^2$
Grade of steel	Fe 500
Permissible tensile stress of steel, σ_{st} (N/mm2)	$275 N/mm^2$

Permissible tensile stress in concrete in bending σ_{cbc}(N/mm2)	11.5 N/mm^2
Crest level	-3.5 m
Bottom width on upstream side	6 m
Height of crest on upstream side	3.5 m
Lowest bed level on upstream side	-7 m
High Flood Level, HFL (Reservoir side)	+5 m
Bottom width on downstream side	12.8 m
Height of crest on downstream side	8.5 m
Lowest bed level on downstream side	-12 m
Design water level at sea side (downstream)	+8.8 m
Span of crest	18.8 m

1.2 <u>Horizontal reinforcement</u>

a. Reservoir side (upstream)	
Vertical water load over the crest, F = $0.5\gamma_W H^2$	103.6 kN/m
Moment due to the vertical water load on crest, M= 0.16 $\gamma_w H^3$	1746 kNm/m
Unit weight of water, γ_w	9.8 N/mm^2
m = 280/(3*σ_{cbc})	8.1
k = 1/(1+σst/(m x σ_{cbc}))	0.2
j = 1-(k/3)	0.9
Thickness of weir, d	2 m
Area of reinforcement (Required), A_{st} = (M/(σ_{st} *j*d)	3451 mm^2
Area of reinforcement (Provided), A_{st}	3927 mm^2
Provide 25mm dia bar @ 125mm c/c spacing	

b. Sea side (downstream)	
Vertical water load over the crest, $F = 0.5\gamma_w H^2$	535 kN/m
Moment due to the vertical water load on crest, $M = 0.16\ \gamma_w H^3$	2285 kNm/m
m = 280/(3*σcbc)	8.1
k = 1/(1+σst/(m x σ_{cbc}))	0.2
j = 1-(k/3)	0.9
Unit weight of water, γ_w	9.8 N/mm^2
Thickness of weir, d	4 m
Area of steel (Required), $A_{st} = (M/(\sigma_{st} *j*d)$	2258 mm^2
Area of reinforcement (Provided), $A_{st} = (M/(\sigma_{st} *j*d)$	2945 mm^2
Provide 25mm dia bar @ 160 mm c/c spacing	

1.3 Vertical reinforcement

a. Reservoir side (upstream)	
Horizontal water pressure over the crest,	118 kN/m^2
Moment due to the vertical water load on crest, $M = 0.16\ \gamma_w H^3$	581 kNm/m
m = 280/(3*σcbc)	8.1
k = 1/(1+σst/(m x σ_{cbc}))	0.2
j = 1-(k/3)	0.9
Unit weight of water, γ_w	9.8 m
Thickness of weir, d	2 m
Area of steel (Required), $A_{st} = (M/(\sigma_{st} *j*d)$	1148 mm^2
Area of reinforcement (Provided), A_{st}	1885 mm^2
Provide 20mm dia bar @ 160mm c/c spacing	

b. Sea side (upstream)	
Horizontal water pressure over the crest	204 kN/m^2
Moment due to the vertical water load on crest	5351 kNm/m
m = 280/(3*σcbc)	8.1
k = 1/(1+σst/(m x σ_{cbc}))	0.2
j = 1-(k/3)	0.9
Thickness of weir, d	4 m
Area of steel (Required), Ast = (M/(σ_{st} *j*d)	5287 mm^2
Area of reinforcement (Provided), A_{st}	6434 mm^2
Provide 32mm dia bar @ 125mm c/c spacing	

ENCLOSURE 8

STRUCTURAL DESIGN OF BREAST WALL

Breast wall is designed according to IS 11130- 1984 and IS 456-2000. Vertical breast wall is provided for vertical open gate.

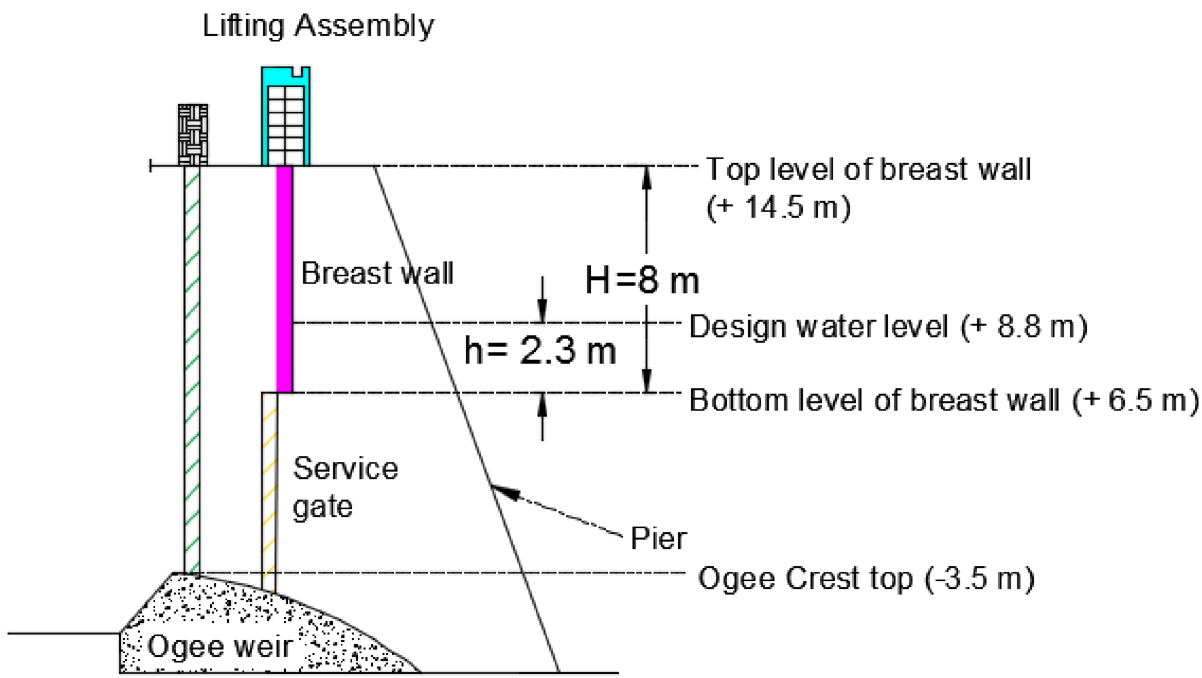

Fig 1 Breast wall

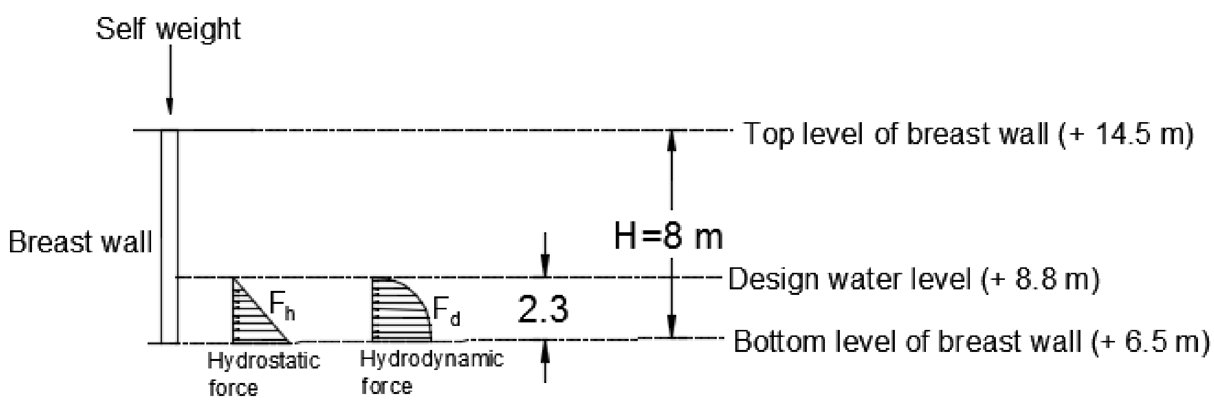

Fig 2 Force diagram of breast wall

2.1 Loads considered

a.	Self-weight of the breast wall	2160 kNm/m
b.	Horizontal water pressure	19 kNm/m
c.	Hydrodynamic forces	7 kNm/m
d.	Seismic loads	562 kNm/m

2.2 Load combinations

a. Water level is at high tide level

b. Water is at high tide level with seismic force

c. Water is at Design flood level

d. Water is at Design flood level with seismic force

2.3 Data used for the design

Compressive strength of concrete, f_{ck}	30 N/mm^2
Grade of steel, f_y	Fe 500
Permissible tensile stress of steel, σ_{st} (N/mm2)	275 N/mm^2
Permissible tensile stress in concrete in bending σ_{cbc}(N/mm2)	10 N/mm2
Top level of breast wall	+14.5 m
Bottom level of breast wall	+6.5 m
Height of breast wall	8 m
Height of water over the breast wall	2.2 m
Design water level in sea side (downstream side)	+ 8.8 m
High flood level (Reservoir side)	+6.5 m
Permissible stress in concrete in bending tension σ_{cbt} (N/mm2)	2 N/mm^2

Clear cover	50 mm
Thickness of breast wall (Assumed)	600mm

2.4 Check for thickness of the wall

Minimum thickness of the wall (mm)	$\sqrt{\frac{6M}{b\ \sigma cbt}} = 280mm$
Moment on wall	26 kNm
Provided thickness of the wall	600mm
Hence, Safe	

2.5 Load combinations

Case 1 - When water level is at high tide level (+6.5 m) without seismic loading	
Moment due to the self-weight, M= γc x Volume of wall x lever arm.	2160 kNm/m
m = 280/(3*σcbc)	9
k = 1/(1+σ_{st}/(m x σ_{cbc}))	0.2
j = 1-(k/3)	0.9
Area of Reinforcement (Required), A_{st} = (M/(σ_{st} *j*d)	1067 mm^2
Minimum area of reinforcement, A_{stmin}, 0.24 % of gross area	1440 mm^2
Area of Reinforcement (Provided), A_{st}	1885 mm^2
Provide 20mm dia bar @ 160mm c/c spacing in both directions	

Case 2- When water level is at high tide level (+6.5 m) with seismic loading in X-direction	
Moment due to self-weight, M= γ_c x Volume of wall.	2160 kNm/m

Horizontal moment due to seismic force, M= α_hx self-weight of wall	562 kNm/m
m = 280/(3*σ_{cbc})	9
k = 1/(1+σ_{st}/(m x σ_{cbc}))	0.2
j = 1-(k/3)	0.9
Unit weight of concrete, γ_c	25 N/mm^2
Seismic coefficient, α_h	0.26 g
Minimum area of reinforcement, A_{stmin}, 0.24 % of gross area	1440 mm^2
Area of horizontal reinforcement (Required), A_{st} = (M/(σ_{st} *j*d)	1067 mm^2
Area of horizontal reinforcement(Provided), A_{st}	1885 mm^2
Provide 20mm dia bar @ 160mm c/c spacing	
Area of vertical reinforcement steel (Required), A_{st} = (M/(σ_{st} *j*d)	278 mm^2
Area of vertical reinforcement steel (Provided), A_{st}	1885 mm^2
Provide 20 mm dia bar @ 160 mm c/c spacing	

Case 3- When water level is at design waterlevel (+8.8 m) without seismic loading in X-direction	
Moment due to self-weight	2160 kNm/m
Horizontal moment hydrostatic pressure	19 kNm/m
m = 280/(3*σcbc)	9
k = 1/(1+σ_{st}/(m x σ_{cbc}))	0.2
j = 1-(k/3)	0.9
Minimum area of reinforcement, A_{stmin}, 0.24 % of gross area	1440 mm^2
Area of horizontal reinforcement (Required), A_{st} = (M/(σ_{st} *j*d)	1067mm^2
Area of horizontal reinforcement (Provided), A_{st}	1885 mm^2
Provide 20mm dia bar @ 160mm c/c spacing	
Area of vertical reinforcement (Required), Ast = (M/(σ_{st} *j*d)	34mm^2

Area of vertical reinforcement (Provided), A_{st}	1885 mm^2
Provide 20 mm dia bar @ 160 mm c/c spacing	

Case 4- When water level is at design water level (+8.8 m) with seismic loading in X-direction	
Moment due to self-weight	2581.2 kNm/m
Horizontal moment hydrostatic and hydrodynamic pressure	587.56 kNm/m
m = 280/(3*σcbc)	9.33
$k = 1/(1+\sigma_{st}/(m \times \sigma_{cbc}))$	0.25
j = 1-(k/3)	0.92
Minimum area of reinforcement, A_{stmin}, 0.24 % of gross area	1440 mm^2
Area of horizontal reinforcement (Required), $A_{st} = (M/(\sigma_{st} *j*d)$	1275 mm^2
Area of horizontal reinforcement (Provided), A_{st}	1885mm^2
Provide 20mm dia bar @ 160mm c/c spacing	
Area of vertical reinforcement steel (Required), $A_{st} = (M/(\sigma_{st} *j*d)$	290 mm^2
Area of vertical reinforcement steel (Required), A_{st}	1885 mm^2
Provide 20 mm dia bar @ 160 mm c/c spacing	

Provide vertical reinforcementand horizontal reinforcement of 20mm dia bar @ 160mm c/c spacing.