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STRUCTURAL DESIGN AND ANALYSIS OFRCC WATER INTAKE STRUCTURE

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ABSTRACT
Renewable energy also termed as green energy, which is the future of the world helps in reducing of tons of kgs of CO2 and hence protecting the environment and meeting the sustainable demand. The major source of renewable energy is through Solar, Wind, and Hydro Projects. There has been a major development in the field of hydro from the last few years and as the potential sites for Hydroelectric project are declining and need for the energy storage is growing, these problems have led to the establishment of pumped storage plants. Pumped storage plants eradicate the problem of potential site as well as energy storage. Pumped storage plant consists of upper and lower reservoir, water conveyance system, **reversible Franci's turbine**, upper and lower intake structures, and tail race channel. Upper and lower/pump Intake is the most important hydraulic component. This is because it determines the performance of the turbines. The main functions of the intake structure are to pump water from the reservoir (upper/lower) to the penstock. The intake structure is equipped with trash rack to avoid the entrance of any considerable size debris which may harm the performance of hydraulic turbines (Impulse/reaction). The water intake structure is provided keeping in mind about hydraulic characteristics of water that is allowable/restricted velocity for smooth transition and to avoid vortices as well as cross flows. The structural design and structural analysis of RCC water intake structure are assure with calculation of safety factors for Sliding, uplift, overturning, stresses in foundation and walls using STADD software.

1 INTRODUCTION

INTAKE STRUCTURES AND TYPES OF

At the point when water is removed through a channel from stream or repository freely and as such the entry of conductors is definitely not a necessary piece of the dam or some other related structure, then an admission structure should be built at the entry of the course. An admission construction might differ from a straightforward substantial block supporting the finish of the course line to tremendous substantial pinnacles, contingent on various factors, for example, repository qualities, limit and release necessities, climatic circumstances, and so on. The essential capability of an admission structure is to help in securely pulling out water from the repository over a foreordained scope of pool levels and in this way to

Fir.1 - RIVER INTAKE

Types of Intake Structures

Several types of intake structure are discussed below:

1) Simple Submerged Intakes

- A straightforward lowered consumption comprises of a basic substantial block, or a stone filled lumber lodging supporting the beginning finish of the withdrawal pipe, as displayed in figure.

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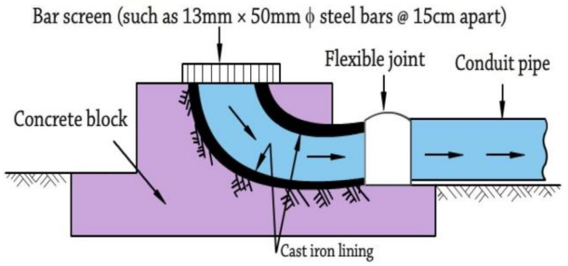


Fig.2 - Simple Concrete Block Submerged Intake

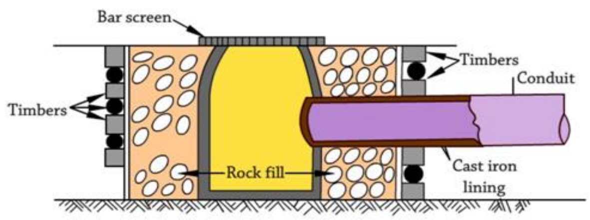


Fig.3 - Rock filled timber Submerged Intake

- Such intake structures should be placed in the river or the reservoir ar a place they may be buried under the sediment.
- Submerged intakes are very economical and do not obstruct navigation.
- It is widely used on small works, and are particularly suitable as water supply intakes from small rivers.
- It is used as intakes to sluice ways of earthen dams with hydraulically operated gate for flow regulation.
- These intkaes are not used on bigger projects as their main disadvantage is the fact that they are not easily accessible for repairing of their gates, etc.

2. LITERATURE REVIEW

The report by **Gregory Fenves & Anik K. Chopra** says that the analysis procedure of the dams must be simplified to show the effects of interaction of dam foundation rock and lower materials of the reservoir. This is done to include it in preliminary stage of design.

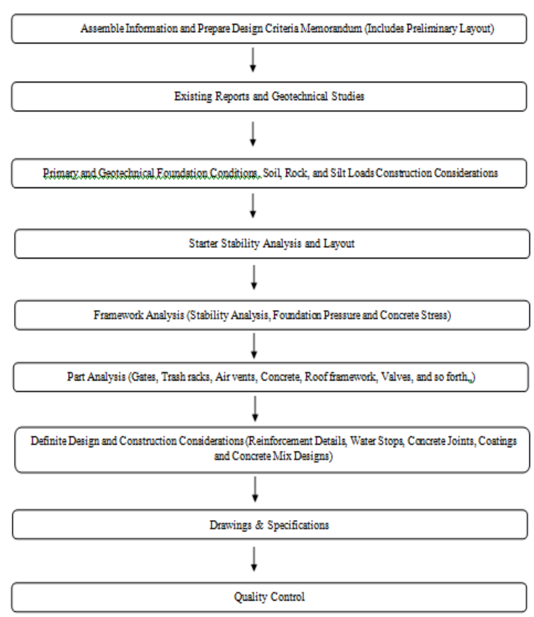
The worked on method showed the calculation of the horizontal tremor powers related with the principal vibration method of the dam.

As per book "Guidelines for design of intakes in Hydroelectric powerplant," the intakes as are provided just after the forebay to avoid the cross flow, vortex and for smooth transition of the flow. While performing the structural design/analysis of intakes all the loads (Geotechnical / RCC / Hydrostatic/ Hydrodynamic loads) acting on the intakes must be considered carefully, and the analysis should be done by identifying the worst-case scenario of the load combination.

K. Safavi.et.al directed pressure driven model review to accumulate information of water powered plan for the proposed consumption structure and to guarantee good water driven execution. The model which was scaled was assessed to figure out the head misfortunes, release limit and its vortex arrangement likelihood which could influence the exhibition of the water powered structures as well as electromechanical gear like siphon and turbines, and so forth. From the aftereffects of the analyses, it was seen that water driven model showed that stream condition along the pinnacle and at admissions is palatable. No stream partition was found at advances and particularly at 90^0 bend of the stream from the admissions. Furthermore, pressures estimated at various focuses were in satisfactory reach. A base water level otherwise called least drawdown level was likewise found for every admission through model trial at which vortex didn't shape in the repository.

In the time of modern turn of events, lakes, streams, and trenches have been taken advantage of to a steadily expanding degree, and dams and weirs for the redirection of stream water have been built on streaming streams for different purposes.

3. METHODOLOGY



4. DESCRIPTION OF STRUCTURE

The structural design is conducted by application STAAD Pro software. The Intake is split into three different

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structures due to thermal cracking requirements. Contraction joints are provided to split the structure into three parts Part I, Part II & Part

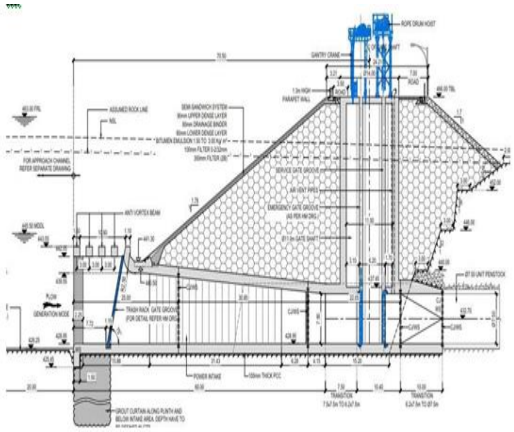

Figure 22: Cross section of the structure

For structural analysis STAAD Pro, v 8i, has been used to model the power intake structure.

The Structure has been modelled using 3 and 4 node plate elements available in STAAD Pro. Staad model has been developed for analyzing the structure for most critical load combination.

The variations in geometry are taken into consideration for deciding the mesh density and element size. In general, mesh size of about 0.5m x 0.5m has been maintained in uniform geometry region. STAAD modelling has been conducted with centerline dimensions of each.

Modelled dimension of structure may slightly vary from actual to avoid badly shaped or warped finite elements.

The dimensional details and a 3D perspective view of structure as modelled in STAAD have been shown below:

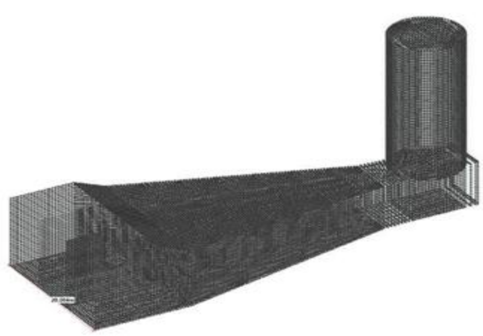

Figure 23: STAAD Model

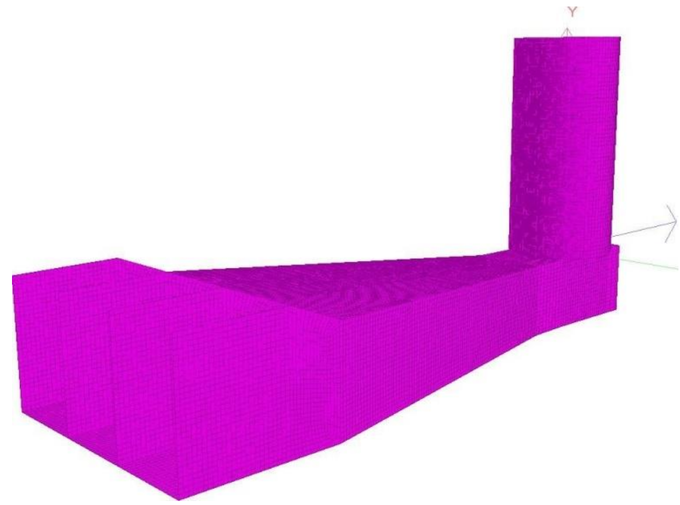

Figure 24: STAAD Render View

4.1 MATERIAL PROPERTIES

Steel		
Grade of structural Steel/UTS/Yield	:	E250/410 MPa/250 MPa
Unit Weight of steel	:	7.850 T/m3
Young's Modulus of Structural Steel	:	2 x 105 N/mm2 [MPa]
Thickness of liner	:	10 mm (Assumed)
Grade of Tor Steel	:	Fe-550 [550 MPa]
Concrete		
Grade of Concrete	:	M25
Unit weight of RCC	:	25.0 kN/m3
Poisson's Ratio of concrete	:	0.17
Young's Modulus of concrete	:	$5000\sqrt{fck}$ MPa
Grade of Reinforcing Steel/Yield	:	Fe 500/ 500 MPa
Clear Cover	:	75 mm
Cover for Crack width in Concrete (mm)	:	45 (mm)
Crack Width in Concrete (mm)	:	0.2 (mm)
Rockfill Dam Profile		
Unit weight of Rockfill	:	24.5 kN/m3
Poisson's Ratio of concrete	:	0.3
Phi of Material	:	40 Degree
Cohesion of backfill	:	0 kPa
Foundation Parameters		
Assumed SBC of Founding Material, SBC	:	900 kN/m2
Factor of safety (assumed) FOS	:	2.5
Modulus of Soil Subgrade reaction (40*Soil Bearing Capacity *FOS)	:	90,000 kN/m2/m

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5. ANALYSIS AND DESIGN OF INTAKE STRUCTURE

PRELIMINARY LOADSDL - Dead load (Self Weight)

- This load is considered by STAAD-Pro automatically based on the geometrical properties of the structure. Inbuilt gravity value of 9.81 m/sec2 is taken for the estimation of the self-weight.
- For the trash rack model dead weight of the vortex beams is added into the model, but its stiffness is notconsidered.
- Dead weight(self-weight) of horizontal beam is about 1.50 x 1.50 x 9.50 x 25 = 534.375 kN for the single piece. Same weight is applied on the middle pier and half of the weight is applied on the end pier.

SL1 - Soil Dead load

As per the stability report material 3A Rockfill is considered as backfill on the top of the intake, and its properties aretabulated under.

For the Trash Rack part Weight of fill material 2 m to 5 m varying height.

Cover	:	2 - 5 meters
Unit weight of concrete material	:	24.5 kN/m3
Load on slab	:	
		49 kN/m2 to 122.5 kN/m2 for the transition part Weight of fill material 5 m to 28.05 m varying height.
Cover	:	5 – 28.05 m
Unit weight of specified RCC material	:	24.5 kN/m3
Load on slab	:	
		122.50 kN/m2 to 687.225 kN/m2 for the gate shaft part. Weight of fill material is taken to be constant for 28.05m height.
Cover	:	28.05 m
Unit weight of specified RCC material	:	24.5 kN/m3
Load on Chamber Slab	:	687.225 kN/m2
Ko of Soil for lateral pressure	:	[1 – Sin {Φ}] : 0.3572

For gate shaft (steel gates) lateral rockfill pressure is variable from top of Dam to the level of the orifice. Total height of the rock mass is about 28.05 m, which gives a maximum lateral rockfill pressure of 28.05 x 24.50 x 0.3572 = 245.476 kN/m2.

WL1 – FRL WATER LOAD [UPLIFT force in FRL Case]

Uplift for at the foundation level is applied on all the three parts as a plate pressure with value as follows:

FRL	:	463.00 m masl
Foundation level	:	426.55 m masl
Unit weight of water	:	9.81 kN/m3
Total Head	:	36.45 m
Total Uplift Pressure	:	357.60 kN/m2

WL2 - FRL WATER LOAD [GRAVITY] - GATE OPEN

Except the Uplift, all the other water loads are taken under this head, during generation and pumping mode and water inthe reservoir at FRL.

For the Trash Rack Model all walls are in balance condition, as water is present on both the side under all operation of theIntake.

For transition part inner walls are in balance water force condition and internal water force is applied on the external walls. For balancing of this force, compression only support system is considered on the external wall of the intake.

For gate shaft model on outer RCC walls water pressure is applied on RCC walls, and it is considered that it will be balanced by the compression only support system applied on the wall.

All the parts internal water pressure is applied on the roof and slab, depending upon the depth from the FRL.

FRL	:	463.00 m masl (meter above sea level)
Invert of Intake	:	428.95 m masl (meter above sea level)
Pressure of Foundation on concrete slab:		34.05 x 9.81 = 334.03 kN/m3
Unit weight of water (H2O)	:	9.81 kN/m3

For the Trash Rack part most of the roof is open and for the partial part that is closed below mention pressure are applied.

FRL	:	463.00 m masl
Overt of Intake	:	439.56 m masl
Uplift on Top Slab	:	23.44 x 9.81 =229.94 kN/m3
Top of Pier at Slab Location	:	441.30 m masl
Top Pressure on Top Slab above vertical rcc walls	:	21.70 x 9.81 =212.87 kN/m3

As the entire transition part in submerged and is supporting the rockfill, it is top slab will be having water load form the top and as well as form the bottom as uplift. The values for these pressures are as follows:

FRL	:	463.00 m masl
Overt of Intake at start	:	438.50 m masl
Uplift on Top Slab at Start	:	24.50 x 9.81 =240.34 kN/m3
Overt of Intake at end	:	436.45 m masl
Uplift on Top Slab at End	:	26.55 x 9.81 =260.45 kN/m3
Water Elevation at Start	:	446.15 m masl
Down Pressure on Top Slab at Start	:	16.85 x 9.81 =165.30 kN/m3
Water Elevation at End	:	463.00 m masl
Down Pressure on Top Slab at End	:	0 x 9.81 =0 kN/m3
Elevation of Wall at Start	:	438.50m masl
Pressure on wall at start	:	24.50 x 9.81 =240.34 kN/m3
Elevation of Wall at End	:	436.45 m masl
Pressure on wall at End	:	26.55 x 9.81 =260.45 kN/m3
Base of wall	:	428.95 m masl
Pressure at base of wall	:	34.05 x 9.81 = 334.03 kN/m3

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The whole gate shaft is embedded in rockfill which will be unsaturated. Hence, there will be no water pressure that will be applied to gate shaft from outer side. Internal pressure corresponding to FRL, and invert of foundation is applied on slab, Walls, orifice and inside lining of shaft.

Values of these pressures are as follows:

FRL	:	463.00 m masl
Invert of Gate Shaft	:	428.95 m masl
Pressure on Foundation & Wall base	:	34.05 x 9.81 = 334.03 kN/m3
Unit weight of water	:	9.81 kN/m3
Top Elevation of Wall	:	436.45 m masl
Pressure on wall at Top	:	26.55 x 9.81 =260.45 kN/m3
Uplift on Chamber Top Slab	:	26.55 x 9.81 =260.45 kN/m3
Top of Orifice	:	438.00 m masl
Pressure on circular wall bottom	:	25 x 9.81 = 245.25 kN/m3
Pressure on circular wall Top	:	0 x 9.81 = 0 kN/m3

EQX - SEISMIC LOAD X Y Z

Based on the Design Based Memorandum Report, The Project lies in the lowest seismic zone -II as per zoning map of India (IS 1893.2022) is in Zone V as per the latest Seismic Zoning Map of India. The horizontal earthquake force or the inertia forces has been determined from site specific seismic recommendations. The horizontal inertial force is calculated by multiplying the seismic coefficient with the weight of the structure. Inertia force will be acting at the centroid of the structure.

For the simplicity of the seismic load directly seismic co-efficient is multiplied with the seismic weight and applied as a force under the head of seismic load.

Hence, Seismic Load in
X = (0.06 x Self-weight Seismic Load in Z) = (0.06 x Self-weight Seismic Load in Y) = (0.04 x Self-weight).
Also, In-built definition of seismic load is also mentioned, for the consideration of variation of seismic co-efficient with respect to the height of the structure. Here, parameters are added such that seismic parameters of the site-specific report can be modelled. For all the calculation of seismic forces, ground level is considered at the foundation level of the Intake.

WL3 & WL4 – Hydro Dynamic Load X & Z

Hydro-Dynamic Load of water is calculated as per the IS Code: 1893-1984. All the aspects of the Design Based memorandum section.
Loads due to earthquake are followed.
In general, pressure zero is taken at FRL and maximum pressure increment is considered at the foundation level. As Part- I and Part-II Trash Rash & Transition are fully submerged, so only the bottom part of the hydro-dynamic increment will be applied.

For the detail values of the pressure refer the Annexure-1

$$p = C_s \alpha_h w h$$

where

p = hydrodynamic pressure in kg/m² 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/m³, and
h = depth of reservior in m.

$$C_s = \frac{C_m}{2}\left\{\frac{y}{h}\left(2-\frac{y}{h}\right)+\sqrt{\frac{y}{h}\left(2-\frac{y}{h}\right)}\right\}$$

where

C_m = maximum value of C_s
y = depth below surface, and
h = depth of reservoir.

SL1 & SL2 – Soil Dynamic Load X & Z

Soil-Dynamic Load of rock mass backfill is calculated as per the IS Code: 1893-1984. All the aspects of theDesign Based memorandum section. Loads due to earthquake are followed.
For the Part-I & II, soil is not laterally restrained by the structure, hence dynamic increment is not considered.As these two parts are only having the soil backfill as load, vertical seismic co-efficient times the weight of the backfill is taken to be seismic load in X & Z.

For the Part III, soil is acting as backfill and laterally restraining the structure. Hence, seismic mass times the vertical seismic co-efficient is taken to be seismic load in X & Z directions.

In general, for the shaft, zero pressure is taken to be at foundation level and dynamic increment is calculated considering the FRL as the top of the Structure. Soil dynamic increment is applied as average in three parts, as the incremental pressure is not significant.

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$$P_a = \frac{1}{2} w h^2 C_a$$

where

P_a = active earth pressure in kg/m length of wall,

w = unit weight of soil in kg/m³,

h = height of wall in m, and

$$C_a = \frac{(1 \pm \alpha_v)\cos^2(\phi - \lambda - \alpha)}{\cos\lambda \cos^2\alpha \cos(\delta + \alpha + \lambda)} \times \left[\frac{1}{1 + \left\{\frac{\sin(\phi+\delta)\sin(\phi - \iota - \lambda)}{\cos(\alpha - \iota)\cos(\delta + \alpha + \lambda)}\right\}^{\frac{1}{2}}}\right]^2$$

the maximum of the two being the value for design,

α_v = vertical seismic coefficient — its direction being taken consistently throughout the stability analysis of wall and equal to $\frac{1}{2}\alpha_h$

ϕ = angle of internal friction of soil,

$$\lambda = \tan^{-1}\frac{\alpha_h}{1 \pm \alpha_v}$$

α = angle which earth face of the wall makes with the vertical,

ι = slope of earthfill,

δ = angle of friction between the wall and earthfill, and

α_h = horizontal seismic coefficient

IPL – IMPOSED LOADS

Under this header, live loads, loads from machinery and H&M loads are considered.

For the Part-I weight of the Trash Rack Gate and its resistance to flow is considered. During the construction it was envisaged that no load of equipment will be transferred to the structure. Please note the tentative loads that will be transferred by the gate are considered in this report. However, detail loads will be given in the design report of the gate. For the fixation of our structure these loads are taken to be on conservative side.

For the Part-II, during the construction it is envisaged that, construction equipment like rollers and compactors are going to transfer the direct and vibration pressure on the top slab of the intake. As per IRC: 6 surcharges of 24 kPa are considered on the top, as an equivalent load.

Dead Weight of Trash Rack Gate	:	[45 Tones] 441.45 kN
Size of Gate	:	8.0 m [W] x 13.10 m [H]
Clogging of Gate	:	30%
Differential Head of Design	:	7.0 m
Horizontal Force	:	7.0 x 9.81 x 8.0 x 13.10 x 0.30
	:	2158.98 kN
Perimeter of Gate	:	2 x [14.50 + 9.0] = 47.0 m2
Horizontal thrust of Gate	:	45.935 kN/m
Element Size	:	0.50 m
Horizontal Point Load	:	22.96 kN
Vertical Point Load	:	[441.45 / 8] x 0.50 = 27.59 kN

For the Part III besides the surcharge of 24 kPa on the top of the shaft and Chamber slab top, its lateral effect on the shaft is also taken. This comes to be 0.3572 x 24 = 8.57 kPa, this is applied uniformly on the walls of the shaft. As per the H&M weight of the stop-log gate and Main Gate, with loads of Fixed Gantry and moveable gantry are applied in the model.

Dead Weight Stop Log Gate	:	[40 Tones] 392.40 kN
Dead Weight Main Gate	:	[55 Tones] 539.55 kN
Dead Weight Fixed Gantry	:	[72 Tones] 706.32 kN
Dead Weight Moving Gantry	:	[35 Tones] 343.35 kN
Dead Weight of Stop Log Gate	:	[40 Tones] 392.40 kN
Size of Gate	:	6.2 m [W] x 7.5 m [H]
Differential Head of Design	:	26.50 m to 34.05 m
Horizontal Force	:	[26.50+34.05]x0.5x9.81x6.2x7.5
	:	13810.40 kN
Nos of Loading Points	:	7.20 x 2 / 0.50 = 28 Nos
Horizontal thrust of Gate	:	493.23 kN
Vertical Point Load	:	[392.40/2] = 196.20 kN

TWL – TRANSIENT WATER LOAD

As per the Transient report of turbine, maximum water head on the turbine will be 517.65 masl, which is 750 m away from the Intake Start. Considering the uniform pressure rise we get a pressure rise of about 6.56 m at distance of 90 m form the start of the intake and below the gate shaft.

This head is uniformly applied in the Part-III model of the gate shaft and uniformly from 0 to 6.56 m of water head in the Part-II of the intake that is Transition. For the Part I considering to be at 23.80 m from start, we get a pressure rise of about 1.734 m this is applied as a constant pressure at the roof of the structure.

WL5 - MDDL WATER LOAD [GRAVITY] - GATE OPEN

Under this condition all the water load as per the load is applied, only instead of FRL. MDDL is considered for the calculation of water head.

WL6 - FRL WATER LOAD [GRAVITY] - GATE CLOSED

As the Part-I & II are fully submerged and are before the stop log gate, this load is same as the load, considering the water at FRL. Only for the gate shaft model Part-III this load is changed, considering that no water and its loads will be present at the back of the stop log gate and in the shaft.

WL7 - FRL WATER LOAD [UPLIFT IN MDDL CASE]

This load is only present in the Part-I model that is Trash Rack as, during the sudden drawdown condition the uplift will not be equal to FRL or MDDL. Study of the uplift for

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the Part-I is present in Annexure-3.

On the conservative side during the sudden drawdown, uplift for the Part-II & III is considered with respect to FRL and this load is present as above mentioned.

During construction only self-weight and rock mass backfill load are envisaged. For this serviceability load case are asfollows:

LOAD 200 LOADTYPE No TITLE 1.00DL+1.00SL

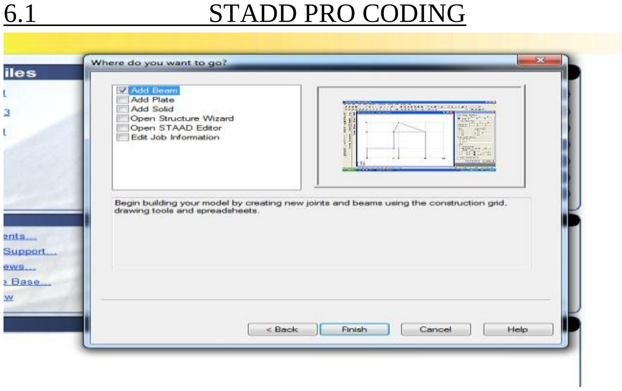
Operating Condition Load

During Operation load of backfill, water and imposed live loads are also taken besides the dead weight of the structure.For this mode of operation of plant load cases are as follows:

LOAD 205 LOADTYPE None TITLE1.00DL+1.00SL+1.00WLLOAD 208 LOADTYPE None TITLE1.00DL+1.00SL+1.00IPL
LOAD 211 LOADTYPE None TITLE1.00DL+1.00SL+1.00IPL+1.00WLLOAD 218 LOADTYPE None TITLE1.00DL+1.00SL+1.00WLGC

CHAPTER-6

6.1 STADD PRO CODING



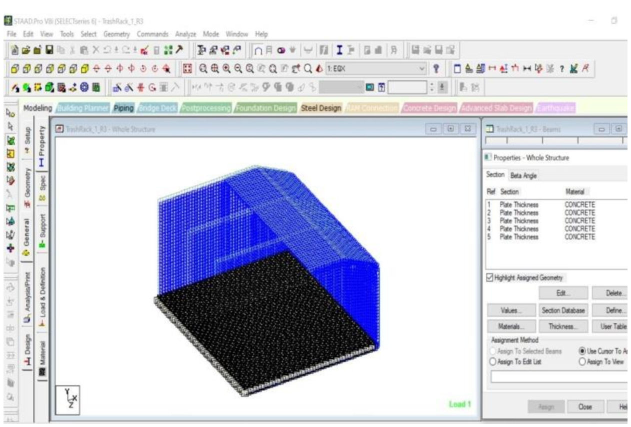


Fig.27 - BOTTOM SLAB AND SIDE WALLS (TRASH RACK) – STADD MODEL IMAGE

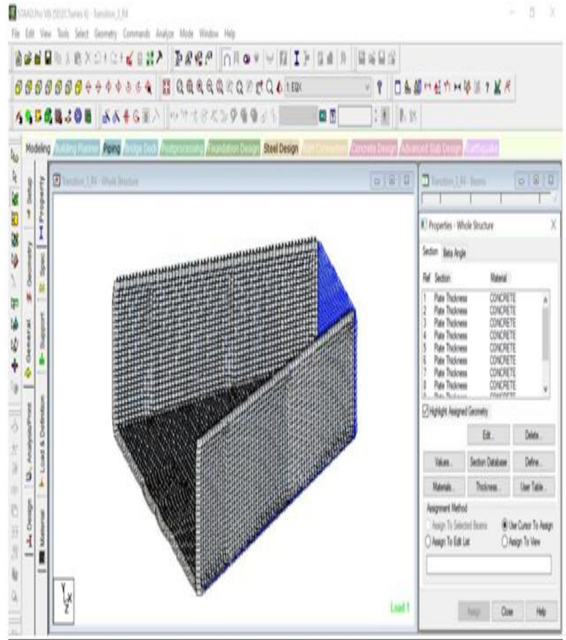


Fig.28 - TRANSITION PART (RECTRANGULAR TO CIRCULAR) – STADD MODEL IMAGE

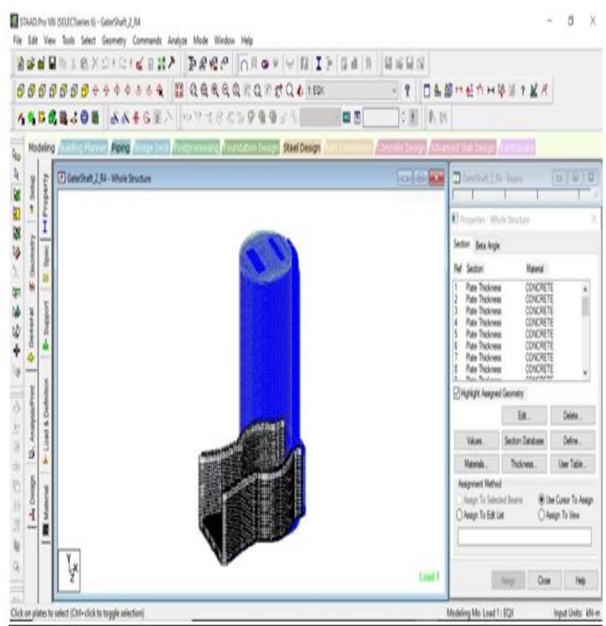


fig.29 - gate shaft (vertical tower) – stadd model image

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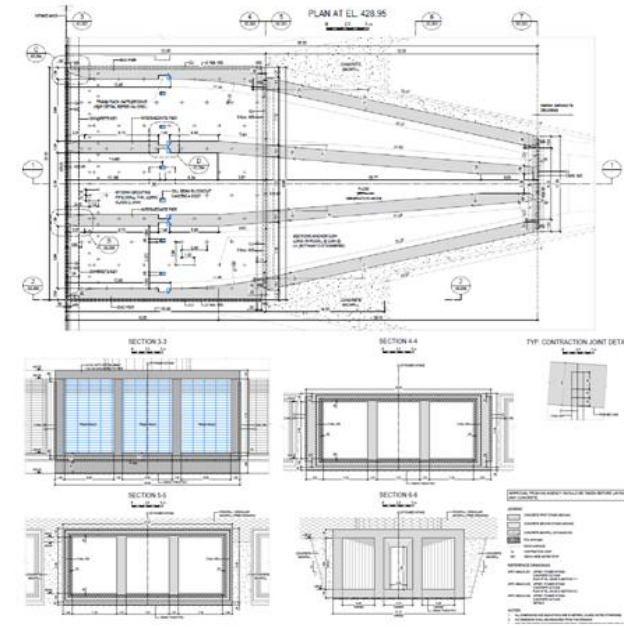
7.1DRAWINGS AND DETAILING



fig,30 - general arrangement details – plan & cross sections

PLAN & LONGITUDINAL SECTIONS

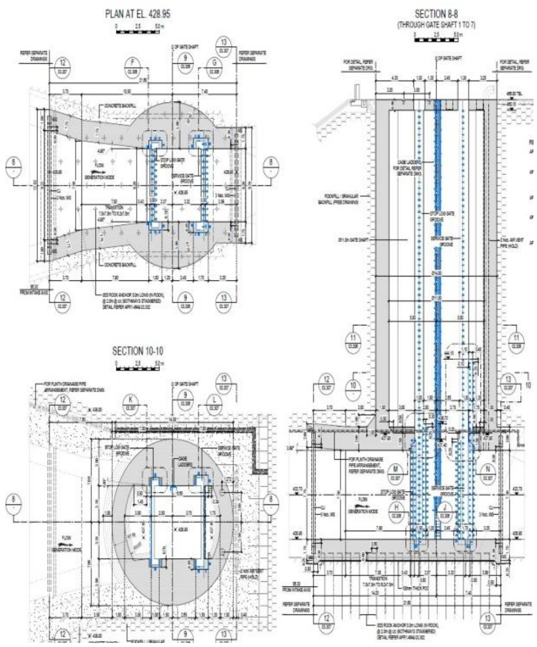


fig.32 - general arrangement details

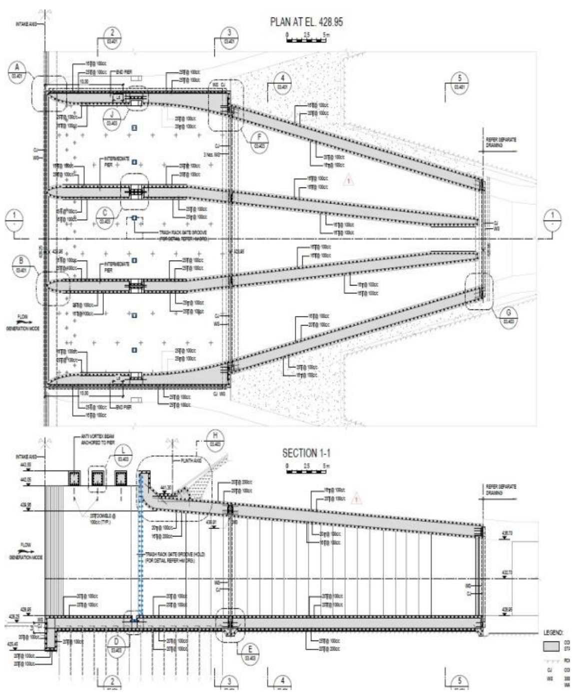


Fig.31 - REINFORCEMENT DETAILS –

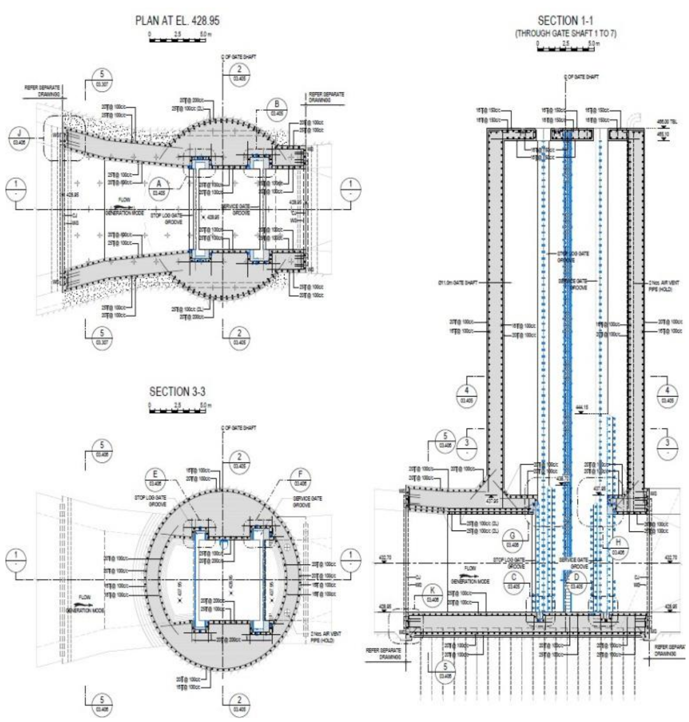


fig.33 - reinforcement details

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8.SUMMARY OF REINFORCEMENTS

Detail calculation of reinforcement and stability is present in following Annexures.

Part I			Dia	Spacing	Layers	Reinforcement
Base Raft	Short Span	Top	25	100	1	25 Dia 100 c/c 1- Layer
Base Raft	Short Span	Bottom	20	100	1	20 Dia 100 c/c 1- Layer
Base Raft	Long Span	Top	16	100	1	16 Dia 100 c/c 1- Layer
Base Raft	Long Span	Bottom	16	100	1	16 Dia 100 c/c 1- Layer
Roof Steel	Short Span	Top	25	100	1	25 Dia 100 c/c 1- Layer
Roof Steel	Short Span	Bottom	20	100	1	20 Dia 100 c/c 1- Layer
Roof Steel	Long Span	Top	16	100	1	16 Dia 100 c/c 1- Layer
Roof Steel	Long Span	Bottom	16	100	1	16 Dia 100 c/c 1- Layer
Pier Free End	Short Span	Inner	25	100	1	25 Dia 100 c/c 1- Layer
Pier Free End	Short Span	Outer	25	100	1	25 Dia 100 c/c 1- Layer
Pier Free End	Long Span	Inner	16	100	1	16 Dia 100 c/c 1- Layer
Pier Free End	Long Span	Outer	16	100	1	16 Dia 100 c/c 1- Layer
Pier Support End	Short Span	Inner	25	100	1	25 Dia 100 c/c 1- Layer
Pier Support End	Short Span	Outer	25	100	1	25 Dia 100 c/c 1- Layer
Pier Support End	Long Span	Inner	25	100	1	25 Dia 100 c/c 1- Layer
Pier Support End	Long Span	Outer	25	100	1	25 Dia 100 c/c 1- Layer
Groove 500 mm THK.	Short Span	Inner	25	100	1	25 Dia 100 c/c 1- Layer
Groove 500 mm THK.	Short Span	Outer	25	100	1	25 Dia 100 c/c 1- Layer
Groove 500 mm THK.	Long Span	Inner	16	100	2	16 Dia 100 c/c 2- Layer
Groove 500 mm THK.	Long Span	Outer	16	100	2	16 Dia 100 c/c 2- Layer
Groove 1000 mm THK.	Short Span	Inner	25	100	1	25 Dia 100 c/c 1- Layer
Groove 1000 mm THK.	Short Span	Outer	25	100	1	25 Dia 100 c/c 1- Layer
Groove 1000 mm THK.	Long Span	Inner	16	100	2	16 Dia 100 c/c 2- Layer
Groove 1000 mm THK.	Long Span	Outer	16	100	2	16 Dia 100 c/c 2- Layer

Part II			Dia	Spacing	Layers	Reinforcement
Base Raft	Short Span	Top	25	100	1	25 Dia 100 c/c 1- Layer
Base Raft	Short Span	Bottom	25	100	1	25 Dia 100 c/c 1- Layer
Base Raft	Long Span	Top	20	100	1	20 Dia 100 c/c 1- Layer
Base Raft	Long Span	Bottom	16	100	1	16 Dia 100 c/c 1- Layer
Roof Steel	Short Span	Top	20	100	1	20 Dia 100 c/c 1- Layer
Roof Steel	Short Span	Bottom	20	100	1	20 Dia 100 c/c 1- Layer
Roof Steel	Long Span	Top	16	100	1	16 Dia 100 c/c 1- Layer
Roof Steel	Long Span	Bottom	16	100	1	16 Dia 100 c/c 1- Layer
Pier Outer	Short Span	Inner	20	100	1	20 Dia 100 c/c 1- Layer
Pier Outer	Short Span	Outer	20	100	1	20 Dia 100 c/c 1- Layer
Pier Outer	Long Span	Inner	16	100	1	16 Dia 100 c/c 1- Layer
Pier Outer	Long Span	Outer	16	100	1	16 Dia 100 c/c 1- Layer
Pier Inner	Short Span	Inner	16	100	1	16 Dia 100 c/c 1- Layer
Pier Inner	Short Span	Outer	16	100	1	16 Dia 100 c/c 1- Layer
Pier Inner	Long Span	Inner	16	100	1	16 Dia 100 c/c 1- Layer
Pier Inner	Long Span	Outer	16	100	1	16 Dia 100 c/c 1- Layer

9 ASSUMPTIONS

For the analysis below mention assumptions are made based, for simplicity of the design with being conservative on saferside.

Assumptions

All the elements are designed for bending + shear, as the axial [Compression or Tension] stress in all the three parts is found to be in limit of 0.04xfck which as per the IS Code 456 can be ignored.

Crack width check is done with Axial + Bending. As per the IS Code if compression is less than 0.2xfck then axial force can be ignored for the crack width check. Same for the tension, if tension stress is less than 0.04xfck then tension force can be ignored in the crack width check. As per the above clause check of cracking as per the IS Code 456 Appendix F is being conducted.

For the shear design beam theory is applied, for the capacity of the concrete section Table 19 of IS code 456 is used and then further section is design as per the clause 40 [Limit state of collapse: Shear] of IS Code 456 is used.

For Part III Gate Shaft model, being circular it will be having hoop stress. A separate check for the hoop stress with Axial + Bending for the reinforcement and for the crack is done.

For Grooves already there is a cover of about 250 mm more as 2nd stage concrete. Hence, cover for the durability 45 mm is provided and for the crack width check it is taken to be 30 mm. However, the limit of the crack width is taken to be same as 0.20 mm as per the design basis report. For Grooves required steel for bending + shear is less and ductile than given steel. This is done so that same steel of column can be run symmetrically in the entire pier. Design section at the clear span between the support is taken for bending and for shear further section at effective depth distance is taken for the shear. It shall be noted that entire model is central line model, and forces at the end or at the support are very conservative to design. This approach is also conservative, as at all location sections taken are at more distance than the clear span as per the geometry.

As Part III & II are going to confined in the back fill concrete between the rock, there stability in overturning and sliding is assumed to be safe and only check for the floatation is done. And as Part I is going to anchored with rock in foundation, so it is also assumed to be safe and only check of floatation is done.

DESIGN OBSERVATIONS

It is recommended that rock anchors in the foundation shall be given for all the three parts, for greater FOS against flotation and for reduction in base pressure.

For the stability of the foundation of Part I II & III SBC of 90 T/m2 is assumed and found to be safe, same shall be ensure at the site before

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construction.

Two-layer water stop shall be given at every interface of movement joint. Hydrophilic bar + PVC water stop shall be given at every construction joint. As active water proofing shall be given specially is Part II & III, to seal the micro cracks during the construction phase, this water proofing shall be regularly maintained.

10RESULTS

As it was discussed the entire model is cut into three parts, at the location of the movement joint. This was done for the ease of analysis and to control the thermal cracking.

Admission ought to be adequately weighty so it may not begin drifting because of upthrust of water. Likewise, a weighty admission won't be washed away by weighty water flows. Every one of the powers which are supposed to chip away at admission ought to be painstakingly broke down and admission ought to be intended to endure this multitude of powers. The underpinning of the admission ought to be taken adequately profound. This willtry not to upset of the construction.

Admission ought not be developed in a route channel as could be expected. On the off chance that it should be built it ought to be safeguarded by bunch of heaps all over from powers brought about by moving boats and liners. Sifters as wire cross section ought to be given on all the admission deltas.

This will keep away from section of enormous drifting articles and fishes into the admission. Admission ought to be of such size thus found that enough water can be gotten from the admission in all conditions.

11 CONCLUSION

In this project, the overturning of the structure represents the lowest safety factor of the global stability of the structure. It was verified for each static scenario the tension on the foundation is always compression.

Throughout this project, the crack width was the main problem for most the elements analyzed.

The thickness of the substantial cover molded the break width confirmation, as a higher cover requires a greater support region, while in other global regulation in compel, utilizing less support areas is conceivable.

The finite element model made in STADD PRO was a crucial tool to evaluate the behavior of this structure,even with the geometric simplifications that were made. In a further phase of the project, it would be interesting to proceed a more complex non-linear analysis considering the construction phases, a seismic analysis and to study the imposed deformations on the structure and cracking due to shrinkage and creep phenomenon, as it was just indirectly considered the minimum shrinkage and temperature reinforcement mentioned.

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