BEFORE THE NATIONAL GREEN TRIBUNAL, PRINCIPAL BENCH

NEW DELHI

O.A. No. 485/2023

In the matter of:

Diwan Singh ....Applicant

Versus

State of Uttarakhand and Ors. ....Respondents

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Through

Advocated for the Respondent No.5

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Date - 11/10/24
New Delhi

(1)

BEFORE THE NATIONAL GREEN TRIBUNAL, PRINCIPAL BENCH

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O.A. No. 485/2023

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SHORT COMPLIANCE AFFIDAVIT ALONG WITH REPORT ON BEHALF OF PROJECT PROPONENT i.e., RESPONDENT NO.5 (KHUTANI POWER PROJECTS)

I, Santosh Thakur, S/o Sh. Ram Sagar Thakur, aged about 46 years, Authorized Representative of the project Proponent i.e., Respondent No.5 (Khutani Power Projects) having registered office at 720 Mahabir Prasad Block, Asiad Village, New Delhi – 110049, being well conversant with the facts of the this case, do hereby solemnly affirm and state on oath as hereunder;

1. That I am writing in compliance with the Hon'ble tribunal directive dated 16.07.2024, wherein Respondent No. 5 i.e., Khutani Power Project Limited (referred as Project Proponent) were instructed to recompute the structural design calculations of the constructions in light of the Expert Joint Committee report dated 11.07.2024, and to take necessary remedial and corrective measures accordingly in respect of designing and construction in question. Copy of the order dated 16.07.2024 is hereby annexed as **R/1**.

2. That Tata Consulting Engineers Limited is the project consultant proving consultancy services for preparation of tender documents and detailed design and engineering for execution of the works for Khutani Small Hydro-Electric Project (21 MW) in the state of Uttarakhand.

The Seal of Oath Commissioner
Sl. No. 73/05
NEELAM SHARMA
App. Govt. of
NCT of Delhi
★ New Delhi ★

(2)

3. That Tata Consulting Engineers Limited had undertaken the exercise of reviewing the structural design calculation in light of Expert Joint Committee Report dated 11.07.2024 and had duly complied with the order of this Hon'ble tribunal dated 16.07.2024. The new revised structural design calculation for Tunnel intake undertaken in August 2024 by T.C.E.L. carrying out the necessary changes is enclosed with this affidavit and has been annexed as **R/2**.

4. That the Expert Joint Committee Report dated 11.07.2024 has been accepted and needful exercise of revision of structural design has been freshly undertaken in the month of August, 2024 by T.C.E.L. in terms of the order of this Hon'ble Tribunal dated 16.07.2024, same may be accepted and this report may kindly be taken as part of official record.

D/3113/18

I identified the deponent who has signed in my presence

DEPONENT

VERIFICATION:

11 OCT 2024

Verified at New Delhi on this ____ day of October, 2024 that the contents of my above affidavit are true and correct to my knowledge, no part of its false and nothing material has been concealed therefrom.

DEPONENT

The Seal of Oath Commissioner
Sl. No. 73/05
NEELAM SHARMA
App. Govt. of NCT of Delhi
New Delhi

Solemnly sworn before me read over & explained to the deponent Admitted to be correct

Oath Commissioner, New Delhi

11 OCT 2024

11 OCT 2024

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Item No. 08 Court No. 2

BEFORE THE NATIONAL GREEN TRIBUNAL
PRINCIPAL BENCH, NEW DELHI

Original Application No. 485/2023

Diwan Singh Applicant

Versus

State of Uttarakhand Respondent

Date of hearing: 16.07.2024

CORAM: **HON'BLE MR. JUSTICE SUDHIR AGARWAL JUDICIAL MEMBER**
HON'BLE DR. AFROZ AHMAD, EXPERT MEMBER

Applicant: Applicant in Person (through VC)

Respondents: Ms. Anjali Rajput, Advocate for State of Uttrakhand (through VC)
Mr. Mukesh Verma, Advocate for UKPCB (through VC)
Adv Atif Suhrawardy for the CPCB (through VC)
Mr. Alok Singh, Mr. Deepak Shukla and Ms. Ananya Singh, Advocates for Project Proponent.
Mr. Shamshad, Adv, Amicus Curiae with. Mr. Arijit Sarkar and Ms. Nabeela Jamil, Advocates.

ORDER

1. This Original Application was registered on a letter petition raising complaint that a tunnel, length 1.5 km, is being constructed by M/s Khutani Power Project (hereinafter referred to as **'Project Proponent/PP'**) in village Batgeri and Sirsauli, Tehsil Ganai Gangoli, Block Gangolihat, District Pithoragarh, (State of Uttarakhand). On account of the aforesaid work undertaken by PP, cracks have appeared in houses of residents of locality which is likely to cause damage to lives and property anytime.

2. Taking note of the said complaint, vide order dated 04.08.2023, this Tribunal constituted a Joint Committee comprising representative of Ministry of Environment, Forest and Climate Change (hereinafter referred to as **'MoEF&CC'**), Integrated Regional Office, Dehradun, Uttarakhand,

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Uttarakhand Environment Protection and Pollution Control Board (UEPPCB) and District Magistrate, Pithoragarh. The said Committee was directed to submit a factual Report on the subject.

3. Pursuant thereto, Joint Committee submitted its Report by e-mail dated 18.11.2023. Report said that it could not be ascertained that cracks in some residential buildings in villages have occurred due to any activity of Project Proponent. However, Committee recommended that for finding out reasons of development of cracks, matter may be required to be examined by some subject expert institutes.

4. Consequently, vide order dated 30.01.2024, Tribunal constituted another Expert Committee comprising (i) Director of Wadia Institute of Himalayan Geology, Dehradun, (ii) Director of Institute of Seismological Research, Gandhinagar, (iii) Central Pollution Control Board (iv) Regional Officer, Ministry of Environment, Forests and Climate Change, Dehradun and (v) Collector, Pithoragarh.

5. The said Committee submitted Report dated 19.04.2024, and its observations are as under:

> *"Based on the field visit, observations and available information, the Joint Committee submitted the following:*
> *Looking into all the aspects, past reports and on-site inspection, the causes of the development of cracks in the houses at both villages can be summarized as below:*
>
> > *a) The houses are mostly constructed over colluvial deposit without concrete base slab as such the weight of the overburden (i.e. of the houses) must have distributed un-evenly that resulted in the development of cracks and bulging of the walls.*
> >
> > *b) The houses are constructed using small rock slabs without cementing materials, only the outer sides of the walls have been plastered either by cement and sand, or locally available materials (mixtures of mud and twigs and cow dungs). The roof slabs are constructed over the top of loosely*

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placed small slabs along with that of the roof slab have resulted in redistribution of the overburden weight that led to the development of cracks and bulging of the walls in the houses.

c) Based on the field observations and information collected, it can be concluded that at present cracks observed in the houses of the villages Batgeri and Sirsauli's are due to local construction practices, materials used and local site conditions. These are not developed due to works carried out by M/s. Khutani Power Company Ltd. (no.5 at the project construction site).

d) A site specific seismic study may be carried out for the project by institute of repute. The design calculations of various structures shall be based on seismic coefficient arrived at from such study.

e) The boundary wall height needs to be enhanced to about 6.0 ft, toward River Saryu to avoid the spillage of debris from much dump site no.2.

f) Should submit the action plan to local concern authorities for stabilization of all muck dump site appropriately & concern authority required to review the same periodically and verify the compliance.

g) The debris excavated or any disposable material should not be discharged/disposed on the back of River Saryu or at any non-identified places. It must be disposed of as per conditions laid down in the Consent to Establish (CTE). The SPCH must periodically monitor and ensure strict compliance of the CTE issued."

6. While going through the above Report we found that in respect of seismic studies, observations of Expert Committee are as under:

"A report on structural design calculation for tunnel intake prepared by M/s Tata Consulting Engineers Limited is also shared with the committee. The report was reviewed in detail. It was found ***that M/s Tata has wrongly considered the project in Zone IV as per seismic zoning map of India published by BIS. Actually, the project site is located in Zone V. Considering the available documents, it seems the site specific seismic studies were not carried out for the project. The calculations are based on seismic zone factor only.*** *The recommended horizontal seismic coefficient as per Zone V is 0.24 as per National Committee on*

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Seismic Design Parameters (NCSDP) while M/s Tata has considered 0.11 factor for calculations."

7. Respondent 5 has filed its reply dated 14.05.2024 and on the aspect of above observations made by Expert Committee, it has submitted its own Expert's Explanation as under:

"A report on structural design calculation for tunnel intake prepared by M/s Tata Consulting Engineers Limited is also shared with committee. The report was reviewed in detail. It was found that M/s Tata has wrongly taken the project in Zone IV as per seismic zoning map of India published by BIS. Actually, the project site is located in Zone V. Also, it seems that site specific seismic studies were not carried out for the project. The calculations are based on seismic zone factor only. The recommended horizontal seismic coefficient as per Zone V is 0.24 as per National Committee on Seismic Design Parameters (NCSDP) while M/s Tata has considered 0.11 for calculations.

TCE's Reply

Khutani Small Hydro Electric Project is being developed as a low-head 'Run-of-the-River' type development involving a diversion barrage across the Saraju River in the Bageshwar district located in the Kumaon region of Uttarakhand. The project lies in Zone V as per the Seismic Zoning Map of India incorporated in IS: 1893 (Part-1)-2016

The design horizontal seismic coefficient (ah) for project components are calculated according to Indian Standard IS 1893-1984 criteria for earthquake resistant design of structures and as given below as per seismic coefficient method given in clause 3.4.2.3. of IS 1893-1984 $as=\beta*1*a_0$.

Where,

β = *coefficient depending upon the soil foundation system*

I = *factor depending upon the importance of the structure*

ao = basic horizontal seismic coefficient

For Barrage

$\beta = 1$ *(Refer Table 3)*

$I = 3$ *(Refer Table 4)*

ao = 0.08 (Refer Table 2 for Zone V)

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Therefore, design horizontal seismic coefficient (ah) = 1 x 3 x 0.08 = 0.24

This is in line with the recommended value as per zone V as per National Committee on Seismic Design Parameters (NCSDP) considering highest value of importance factor.

Therefore, Vertical seismic coefficient for barrage, av = 2/3 ah = 0.16

Similarly, for Tunnel Intake Importance factor 1 = 1 being categorised as 'other structure' as per Table 4

Therefore,

Design horizontal seismic coefficient for tunnel intake(ah) = 1 x 1 x 0.08 = 0.08

*However, In the structural design report for tunnel intake vide Doc. No.: TCE 7784A-CV-CALC-3028-01 (R2), the Seismic Coefficient (ah) was calculated as 0.11 following the formula ah = z/2*I/R* Sa/g provided in IS 1893 (Part-1)-2016.*

It is pertinent to mention that IS: 1893 (Part-1)-2016 deals primarily of building structure and the provision indicated in the above code for calculating design seismic coefficient is meant for buildings.

For hydropower structure, mainly dam/barrage/intake, horizontal seismic coefficients are generally calculated as per provision given in clause 3.4.2 3 of IS 1893-1984.

Therefore, the method for calculating design horizontal seismic coefficient following IS:1893 (Part-1)-2016 and seismic zone were wrongly adopted in the structural design report for tunnel intake.

However, the value calculated in the report (0.11) is more than the actual value (0.08) calculated following IS 1893-1984 and thus, will not have any impact in the design aspect.

8. The matter was examined by us on 16.05.2024 when learned Amicus Curei, Sh. M.R Shamsad, Advocate stated that as per Expert Committee's report, in respect of seismic zone, there is an apparent error in as much as structural design for tunnel has been calculated by considering the seismic zone-IV though as a matter of fact, area falls in seismic zone-V and

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therefore, value of seismic co-efficient value (denoted as '**ah**')was computed as 0.11 though it would have been different computed by taking seismic zone as V.

9. In view thereof, we direct Expert Committee constituted by this Tribunal vide order 30.01.2024 to look into this aspect of the matter and submit report.

10. Pursuant to order dated 16.05.2024, Expert Joint Committee's report dated 11.07.2024 has been submitted.

11. Report shows that it has considered comments of Dr. Sumer Chopra, one of the members on Expert Joint Committee in reply submitted by project proponent (respondent 5) and his comments are as under:-

> *"Facts on the reply submitted to NGT on seismic parameter IS 1893 is the Indian standard code of practice for "Criteria for Earthquake Resistant Design of Structures." The code has undergone several revisions to incorporate advancements in knowledge and technology related to earthquake engineering.* ***The IS 1893-1984 is the fourth revision while the IS 1893-2016 is the sixth revision.***
>
> *As per the IS code, the horizontal seismic coefficient (ah) for a structure can be computed by two methods,*
>
> *(i) Seismic coefficient method / Equivalent static method and*
> *(ii) Response spectrum method.*
>
> ***The seismic coefficient method is a simplified approach for seismic analysis,*** *often used for regular and low-rise structures. It approximates the effects of an earthquake by applying equivalent static horizontal forces to the structure.*
>
> ***The response spectrum method is a more refined and accurate approach for seismic analysis, suitable for all types of structures, especially irregular and high-rise buildings.*** *It uses the structure's dynamic characteristics to estimate the seismic response.*
>
> *In IS 1893-1984, the expressions for ah in seismic coefficient method and response spectrum method are different.* ***In IS 1893-2016,***

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although the expression is same, design acceleration coefficient for different soil types, normalized with peak ground acceleration (Sa/g) is different corresponding to natural period (T) of structure. *For example, as per IS 1893-1984, in seismic coefficient method the design value of horizontal seismic coefficient*

$$a_{\beta} \cdot I \cdot a_0 \qquad (1)$$

where 1= a factor, called importance factor, depends upon the importance of the structure

β= a coefficient depending upon the soil-foundation system
a_o= basic horizontal seismic coefficient. This value is different for different seismic
zones.

In response spectrum method,

$$a_h = \beta^* I^* F_0^* Sa/g \qquad (2)$$

where F_o is the seismic zone factor and Sa/g is the average acceleration coefficient for appropriate natural period and damping of the structure.

As per IS 1893-2016, the horizontal seismic coefficient
$$a_h = (Z/2)^*(I/R)^*(Sa/g) \qquad (3)$$

where Z= seismic zone factor and R is the response reduction factor. Sa/g is taken from the following:

For use in equivalent static method

$$\frac{S_a}{g} = \begin{cases} \text{For rocky or hard soil sites} & \begin{cases} 2.5 & 0 < T < 0.40\text{ s} \\ \dfrac{1}{T} & 0.40\text{ s} < T < 4.00\text{ s} \\ 0.25 & T > 4.00\text{ s} \end{cases} \\ \text{For medium stiff soil sites} & \begin{cases} 2.5 & 0 < T < 0.55\text{ s} \\ \dfrac{1.36}{T} & 0.55\text{ s} < T < 4.00\text{ s} \\ 0.34 & T > 4.00\text{ s} \end{cases} \\ \text{For soft soil sites} & \begin{cases} 2.5 & 0 < T < 0.67\text{ s} \\ \dfrac{1.67}{T} & 0.67\text{ s} < T < 4.00\text{ s} \\ 0.42 & T > 4.00\text{ s} \end{cases} \end{cases}$$

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For use in response spectrum method

$$\frac{S_a}{g}=\begin{cases}\text{For rocky or hard soil sites} & \begin{cases}1+15T & T<0.10\text{ s}\\ 2.5 & 0.10\text{ s}<T<0.40\text{ s}\\ \dfrac{1}{T} & 0.40\text{ s}<T<4.00\text{ s}\\ 0.25 & T>4.00\text{ s}\end{cases}\\ \text{For medium stiff soil sites} & \begin{cases}1+15T & T<0.10\text{ s}\\ 2.5 & 0.10\text{ s}<T<0.55\text{ s}\\ \dfrac{1.36}{T} & 0.55\text{ s}<T<4.00\text{ s}\\ 0.34 & T>4.00\text{ s}\end{cases}\\ \text{For soft soil sites} & \begin{cases}1+15T & T<0.10\text{ s}\\ 2.5 & 0.10\text{ s}<T<0.67\text{ s}\\ \dfrac{1.67}{T} & 0.67\text{ s}<T<4.00\text{ s}\\ 0.42 & T>4.00\text{ s}\end{cases}\end{cases}$$

M/s Tata Consulting Engineers Limited *in the structural design report for tunnel intake Doc. No. TCH.7784A-CV-CALC-3028-01 (R1),* ***estimated the seismic coefficient (a_h) following the response spectrum method given in IS 1893-2016.*** *While calculation, M/s Tata Consulting Engineers Limited considered the seismic zone factor Z of seismic zone IV, that is 0.24, 1=1, R=1.5, Sa/g =1.358. Therefore*

a_h=(0.24/2)*(1/1.5)*1.358=0.11.

*Now, M/s Tata Consulting Engineers (TCE) Limited mentioned that the a_h value calculated using equation (1) above i.e. a_h= 13*I* a_o, that is using seismic coefficient method as per IS 1893-1984, is 0.08. They estimated the value by taking Importance factor 1=1, 3=1 and basic horizontal seismic coefficient for zone V (a_0)=0.08. They mentioned that a_h = 0.11, calculated in the report, is more than the a_h value calculated by use of seismic coefficient method of IS 1893-1984. Therefore, it will not have any impact in the design aspect.*

It is observed that

1. *The* ***Khutani Small Hydro Electric Project site is in seismic zone-V*** *and M/s Tata Consulting Engineers (TCE) Limited admitted this fact now. The* ***zone factor Z in equation (3) is 0.36 for zone-V.*** *Considering Sa/g=1.358 (corresponding to the natural period of the structure), 1=1.0 and R=1.5 values as considered by M/s Tata Consulting Engineers Limited makes a_h = (0.36/2)*(1/1.5)*1.358 =0.16. So* ***a_h=0.11 is not correct for sites in zone-V, rather 0.16 is the correct value.***

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2. M/s Tata Consulting Engineers Limited mentioned that IS 1893-2016 (Part 1) deals primarily of building structure and the provision indicated in the code for calculating design seismic coefficient is meant for buildings. For hydropower structure, mainly dam/barrage/intake, horizontal seismic coefficients are generally calculated as per provision given in clause 3.4.2.3 of IS 1983-1984. That makes the ah value 0.08 (equation 1), using the equivalent static method. ***It is important to mention here that 'Recommendations for earthquake resistant design of structures' was first published in 1962, and revised in 1966, 1970, 1975 and 1984. Further, in 2002, the Committee decided to present the provisions for different types of structures in separate parts, to keep abreast with rapid developments and extensive research carried out in earthquake-resistant design of various structures. Thus, IS 1893 was split into five parts.*** *The other parts in the series are:*

Part 1: General provisions and buildings

Part 2: Liquid retaining tanks - Elevated and ground supported

Part 3: Bridges and retaining walls

Part 4: Industrial structures, including stack-like structures

Part 5: Dams and embankments (to be formulated)

However, ***it is mentioned in "Foreword" of IS 1893-2016 that this standard (Part 1) contains general provisions on earthquake hazard assessment applicable to all buildings and structures covered in Parts 2 to 5.***

It is obvious that M/s ***Tata Consulting Engineers Limited is comparing a_h value estimated from older and recent IS codes and that too estimated by two different methods. Comparing the seismic horizontal coefficients calculated using IS 18931984 and IS 1893-2016 reveals significant differences due to updates and improvements in the standards over the years.*** *The following Table-1 presents an overview of the main differences in both the version of the codes.*

Table 1: Comparison of factors used for calculation of a_h in IS 1893-1984 and IS 1893-2016

Factor	IS 1893-1984	IS 1893-2016
Seismic zone factor (Z)	The seismic zones were categorized into five zones (I to V) with corresponding zone factors. Zone V had the highest factor of 0.4.	Zones were updated, with Zone V having a factor of 0.36.

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Importance factor (I)	*The importance factor ranged from 1.0 to 3.0 depending on the type of building.*	*More detailed importance factors ranging from 1.0 to 1.5 or higher for critical structures.*
Response reduction factor (R)	*This factor was not explicitly detailed in the 1984 version as it is in the later versions.*	*More detailed and specific to different structural systems, ranging from 1.5 to 5.0 or more.*
Average acceleration coefficient (Sa/g)	*To be considered for appropriate natural period and damping of the structure*	*To be considered for rock/ stiff soil / soft soil sites based on appropriate natural period of the structure*

"

12. Joint Committee agreeing with the said comment, has recorded its summary findings as under:-

"Summary:

- *The formula for ah is similar in concept for both the versions of the IS codes, but uses updated factors and response spectra, leading to different results.* ***Hence, the seismic horizontal coefficients calculated using IS 1893-1984 and IS 1893-2016 are not directly comparable due to significant updates in seismic zoning, importance factors, response reduction factors, and response spectra.***
- *The 2016 version provides a more accurate and detailed approach reflecting current knowledge and practices in earthquake engineering. Consequently,* ***calculations using the 2016 code will generally result in different, and often higher, seismic forces, ensuring improved safety and performance of structures under seismic events.***
- *Justifying the estimations with older provisions is not correct and there should be consistency in estimations.*
- ***In view of the above, the ah value will be 0.16,*** *considering I=1.0, R=1.5, Sa/g= 1.358 and Z (for zone-V)=0.36."*

13. Learned counsel appearing for respondent 5 stated at the bar today that they do not propose to file any objection to above report dated 11.07.2024 and on the other hand, respondent 5 (proponent) is agreeable to recompute structural design calculation in the light of report submitted

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by Expert Joint Committee and take remedial and corrective measures accordingly in respect of designing and constructions in question.

14. In view thereof, we do not find any exigency to keep the matter pending further and dispose of this original application by directing respondent 5 to reconsider the structural design calculation in the light of Expert Joint Committee report dated 11.07.2024 and take necessary remedial and corrective measures for strengthening the constructions in dispute and take all steps for consequential safety measures within a period of three months and submit a compliance report with Registrar General of this Tribunal, who, if finds necessary, may place the matter before Tribunal for further orders.

15. With the above directions/observations, the original application is disposed of.

Sudhir Agarwal, JM

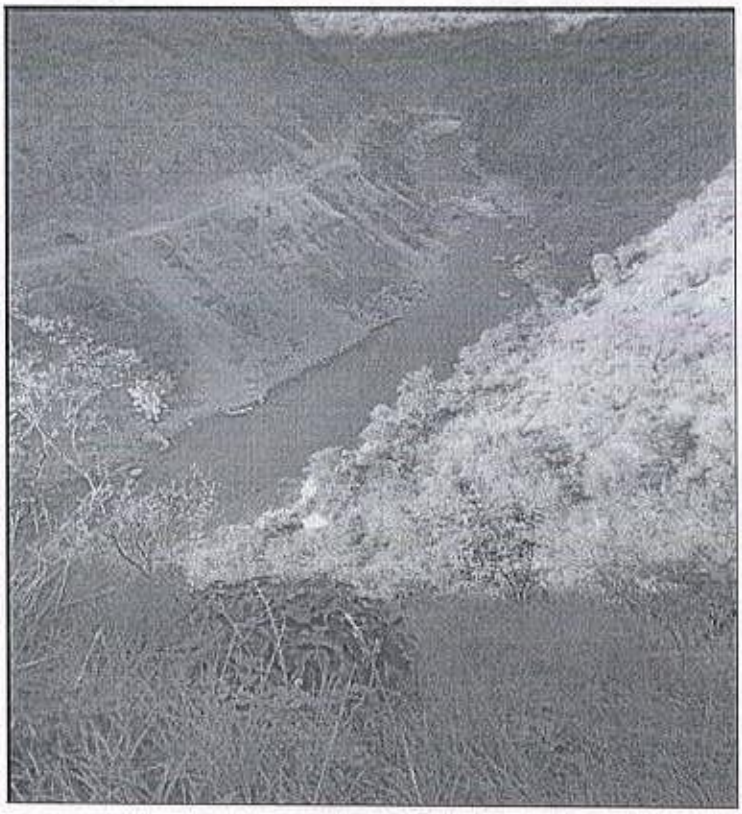
Dr. Afroz Ahmad, EM

July 16, 2024
Original Application No. 485/2023
AB

(14)

Consultancy Services for preparation of tender documents and detailed design and engineering for execution of works for Khutani Small Hydro Electric Project (21 MW) in Uttarakhand

STURCURAL DESIGN CALCULATION FOR TUNNEL INTAKE
August'2024



Khutani Power Company Private Limited (KPCPL)
10, Bankote , Ganaigangoli, Pithorgarh- 262532
Uttaranchal, India

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DOCUMENT RECORD

REVISION STATUS

Doc. No.: TCE.7784A-CV-CALC-3028-01 (R3)

REV. NO.	DESCRIPTION	DATE	PREPARED BY	CHECKED BY	APPROVED BY
R0	Structural Design of Tunnel Intake	29-04-2022	SKS	SP	SKD
R1	Revision made due to modification of steel grade and another minor modification.	13-05-2022	SKS	SP	SKD
R2	Revision made due to change of thickness of raft and pier	24-05-2022	SKS	SP	SKD
R3	Revision made due to change of seismic coefficient	07-08-2024	DC	AHJ	SKD

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1 Introduction

1.1 General

Khutani Small Hydro Electric Project is being envisaged as a low-head 'Run-of-the-River' type development involving a barrage across the Sarju River in the Bageshwar District located in the Kumaon region of Uttarakhand. For power generation, the project shall utilize the flow of the Sarju River by constructing a barrage at about 250 m downstream of the confluence of Bhadargad and Sarju River. Water from the pondage created behind the barrage would be conveyed through a water conductor system comprising of an approach/feeder channel, a Head Race Tunnel (HRT) about 4 km long, surge shaft and penstock. The water would be fed to a surface powerhouse and the tail water discharge from the powerhouse would be released back to the Sarju River.

1.2 Objective of the Report

The present report outlines the design criteria for reinforcement design of Tunnel Intake Structure using IS standard. The structural analysis is done using Bentley Software "STAAD.pro" and design is done using spreadsheet developed as per IS standard.

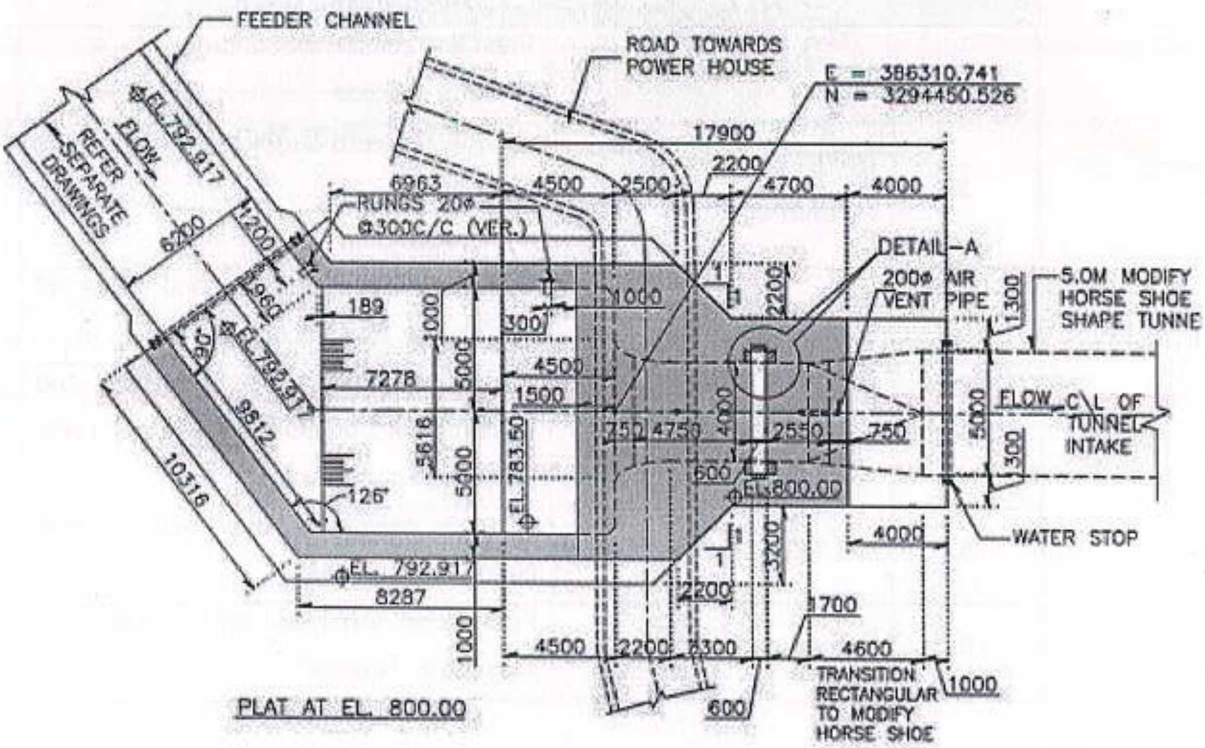

Figure- 1: Plan of Tunnel Intake

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2 Salient Features of Tunnel Intake

S.No.	Description	Unit	Parameter
1	Full Reservoirs Level (FRL)	m	EL. 799.00
2	Maximum Water Level (MWL)	m	EL. 799.00
3	Top Level of Structure	m	EL. 800.00
4	Nos. of Bays	Nos.	One
5	Length	m	14.00
6	Clear Width of each Bay	m	10.00
7	Type of Structure	-	Bell Mouth

3 Standard, Manuals and Other Reference

List of manuals and various Codes are provided in the table below.

S.No.	Code No.	Title
1.	IS-456:2000	Plain and reinforced concrete - Code of practice
2.	IS-1893:(part1) 2016 (reaffirmed 2021)	Criteria for earthquake resistant design of structures
3.	IRC-6-2000	Standard Specifications and code of practice for Road bridges
4.	IS-1904:1986 (reaffirmed 1995)	Code of practice for design and construction of foundation in soils: General requirement
5.	EM-1110-2-2104	Strength design for reinforced concrete hydraulic structure
6.	Reference Book-1	Foundation Analysis and Design, by "Joseph E. Bowles"

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4 UNIT SYSTEM

For all technical documents the SI system will be used. The units of some important parameters are the following:

Description	Unit
Mass	kg
Loads / Forces	kN, kN/m / kN/m^2
Moments (Bending Moments)	kNm
Pressures/ Stresses	N/mm2(=MN/m2 = MPa) or kN/m2 (= kPa)
Volume	m^3
Unit weight	kN/m^3
Angle	Degrees (°)

5 MATERIAL PROPERTIES

5.1 CONCRETE

Grade of concrete	M-25
Characteristic Strength of concrete	25 MPa
Modulus of elasticity	$5000 \times (f_{ck})^{0.5}$
Unit Weight of concrete	25 kN/m^3

5.2 REINFORCEMENT

Grade of Steel	Fe-500
Yield strength of steel	500 MPa
Modulus of elasticity	2×10^5 MPa

5.3 FOUNDATION

Safe Bearing Capacity of Founding Material, SBC 200 kN/m^2 (Assumed)

Sub-grade Modulus, Ks = 2.5*40*SBC 20000 kN/m^2/m

(Note- Sub-grade Modulus has been calculated using Equation 9-9 of "Foundation Analysis and Design" by Joseph E. Bowles @ page no 502; the same value of subgrade modulus has been used in STADD.pro model to simulate soil-structure interaction.)

5.4 EARTH BACKFILL

The below mentioned parameters for Earth backfill has been suitably assumed;

Saturated Unit Weight, γ_{sat} 22.0 kN/m^3

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Cohesion (Considered zero for calculation only, however same shall be as per locally available material at the site) 0.00 kN/m^2

Angle of Friction, φ 30.0 degrees

5.5 WATER

Unit Weight of Water, γ_w = 9.81 kN/m^3

6 DESIGN LOAD

6.1 SELF-WEIGHT

Self-weight of Intake structure has been calculated by STAAD.pro based on geometrical parameters and material properties given as input. This load is permanent action (G).

6.2 SEISMIC INERTIA FORCE (EQX & EQZ)

The design horizontal and vertical seismic coefficients for DBE/OBE condition are recommended as αh = 0.16 and αv= 0.11.

Project is in Zone V as per the Seismic Zoning Map of India incorporated in IS: 1893 (Part-1)–2016 considering project site nearby Almora. The horizontal earthquake force or the inertia forces has been determined from IS Code. 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.

The earthquake forces (both horizontal and vertical) are treated as simple static inertia force and are combined with water or gravity loads.

- Zone factor Z - 0.36
- Importance Factor -1.00
- Response Reduction Factor 1.50

Seismic Coefficient (αh) Shall be calculated as per following formula.

$$\alpha h = Z/2 * I/\mathrm{R} * Sa/g$$

Sa/g is spectral acceleration and calculated corresponding to the natural period of vibration of structure. The natural period of vibration shall be calculated using following formula given in IS 12720:2004.

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The same seismic parameters have been considered in "STAAD.pro" for calculating Pseudo-static seismic inertial forces. Also, for calculating hydrodynamic force, the horizontal seismic coefficient would be 0.16.

6.3 Earth Pressure

Earth pressure up to top of the structure i.e.EL.800.00 have been considered. Saturated unit weight of soil has been considered for calculating the earth pressure. The static component of earth pressure shall be distributed as triangular load with maximum intensity at the base of wall.

During earthquake case, dynamic increment of earth pressure has been calculated as per IS-1893-1984. For calculation, refer **Annexure-I.**

6.4 Water Load

The water load has been applied in two parts viz. Hydrostatic and hydrodynamic. The hydrostatic pressure is simply calculated by multiplying the depth of water with unit weight of water and applied as triangular distribution. The hydrodynamic force has been calculated using Clause 7.2 of IS 1893-1984 and distributed based on the equation for calculating force, for calculation refer **Annexure I.**

The Hydrodynamic Pressure

$$\boxed{p = C_s \alpha_h wh}$$

α_h Design Horizontal Seismic Coefficient
w unit weight of water kN/m^3
h depth of reservoir,m

The Coefficient varies with shape and depth

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

C_m Coefficient Calculated from Fig:10 of IS:1893-1984
y Depth of water from surface of reservoir

6.5 Uplift Pressure

Uplift pressure has been calculated for Full reservoir level (EL.799.00) and act on bottom of the raft.

6.6 Live Load

The Intake slab shall be accessed for inspection and be accessible to vehicle also. As vehicle load have been considered separately, thus, 5 kN/m^2 load intensity has been considered as live load.

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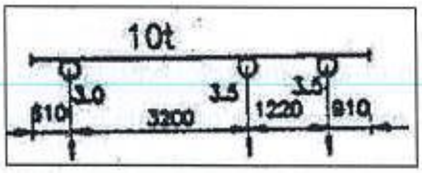
6.7 Vehicle Load on Bridge Slab

As the exact mobile crane (for lifting the gate) which shall be travelled through the bridge slab, is not finalized, 9 R loading (10 ton) from IRC-6 has been assumed for the design.



6.8 Hoist Load

At present hoist load is not conformed by HM designer. However, an assumption has been considered for Hoist load for lifting of gate on main intake as 160 kN which is distributed on four legs.

7 Load Combination

Partial safety factor for material strength in assessing the strength of a structure or structural member for the limit state of collapse, the values of partial safety factor should be taken as 1.5 for concrete and 1.15 for steel (IS 456: 2000).The partial safety factors for various loads and load combinations are as per IS 456:2000 given below.

Load Combination	Limit State of Collapse			Limit State of Serviceability		
	DL	LL	EL	DL	LL	EL
DL + LL	1.5	1.5	-	1.0	1.0	-
DL + EL	1.5	-	1.5	1.0	-	1.0
DL + LL + EL	1.2	1.2	1.2	1.0	0.8	0.8

Note- Second load combination is ignored as live load contribution is found considerably more.

The Primary load for load combination.

a. LC-1 : EQ (+X)
b. LC-2 : EQ (+Z)
c. LC-3 : Dead Load
d. LC-4 : Live Load
e. LC-5A : Hydrostatic Load (Upstream Wall)
f. LC-5B : Hydrostatic Load (Inclined Portion)
g. LC-5C : Hydrostatic Load (Bottom Portion)

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h. LC-5D : Hydrostatic Load On Breast And Backwall
i. LC-6A : External Soil Pressure (Upstream Wall)
j. LC-6B : External Soil Pressure (Inclined Portion)
k. LC-6C : External Soil Pressure (Bottom Portion)
l. LC-7 : Weight of Water
m. LC-8 : Uplift Pressure
n. LC-9 : Hoisting Load
o. LC-10 : Bridge Load
p. LC-11A : Hydrodynamic Loading (+Z)
q. LC-11B : Hydrodynamic Loading (-Z)
r. LC-11C : Hydrodynamic Loading (-X)
s. LC-12A : Dynamic Increment (-Z)
t. LC-12B : Dynamic Increment (+Z)

The structure is analysed for following load combination.

For finalization of General arrangement, concrete definition and reinforcement calculation below mentioned load combination will be used.

101. Operation condition-1: 1.5 LC-3+1.5 LC-4+1.5 LC-7+1.5 LC-5A+1.5 LC-5B+ 1.5 LC-5C+1.5 LC-5D+1.5 LC-8+1.0 LC-9+1.0 LC-10

102. Operation condition-2: 1.5 LC-3+1.5 LC-4+1.5 LC-7+1.5 LC-5A+1.5 LC-5B+ 1.5 LC-5C+1.5 LC-5D+1.5 LC-8+1.0 LC-9

103. Construction condition-1 (Empty intake): 1.5 LC-3+1.5 LC-4+1.5 LC-1.5 LC-6A+1.5 LC-6B+1.5 LC-6C +1.0 LC-9+1.0 LC-10

104. Construction condition-2 (Empty intake): 1.5 LC-3+1.5 LC-4+1.5 LC-1.5 LC-6A+1.5 LC-6B+1.5 LC-6C +1.0 LC-9

105. Operation condition + EQ(+x): 1.2 LC-1+1.2 LC-3+1.2 LC4+1.2 LC-5A+1.2 LC-5B+1.2 LC-5C+1.2 LC-5D+1.2 LC-7+1.2 LC-8+1.0 LC-9+1.2 LC-11C

106. Operation Condition + EQ (+Z): 1.2 LC-2+1.2 LC-3+1.2 LC4+1.2 LC-5A+1.2 LC-5B+1.2 LC-5C+1.2 LC-5D+1.2 LC-7+1.2 LC-8+1.0 LC-9+1.2 LC-11A

107. Operation Condition + EQ (-Z): -1.2 LC-2+1.2 LC-3+1.2 LC4+1.2 LC-5A+1.2 LC-5B+1.2 LC-5C+1.2 LC-5D+1.2 LC-7+1.2 LC-8+1.0 LC-9+1.2 LC-11B

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108. Construction Condition + EQ (-X): -1.2 LC-1+1.2 LC-3+1.2 LC-4+1.2 LC-6A+1.2 LC-6B+1.2 LC-6C+1.0 LC-9

109. Construction Condition + EQ (+Z): 1.2 LC-2+1.2 LC-3+1.2 LC-4+1.2 LC-6A+1.2 LC-6B+1.2 LC-6C+1.0 LC-9+1.2 LC-12B

110. Construction Condition + EQ (-Z): -1.2 LC-2+1.2 LC-3+1.2 LC-4+1.2 LC-6A+1.2 LC-6B+1.2 LC-6C+1.0 LC-9+1.2 LC-12A

For check of deflection, base pressure and stability following load combination has been used.

201. Operation Condition-1 (Serviceability): 1.0 LC-3+1.0 LC-4+1.0 LC-5A+1.0 LC-5B+1.0 LC-5C+1.0 LC-5D+1.0 LC-7+1.0 LC-8+1.0 LC-9+1.0 LC-10

202. Operation Condition-2 (Serviceability): 1.0 LC-3+1.0 LC-4+1.0 LC-5A+1.0 LC-5B+1.0 LC-5C+1.0 LC-5D+1.0 LC-7+1.0 LC-8+1.0 LC-9

203. Construction Condition-1 (Empty Int)-Serviceability: 1.0 LC-3+1.0 LC-4+1.0 LC-5A+1.0 LC-5B+1.0 LC5C+1.0 LC-9+1.0 LC-10

204. Operation Condition-2 (Empty Int)-Serviceability: 1.0 LC-3+1.0 LC-4+1.0 LC-5A+1.0 LC-5B+1.0 LC-5C+1.0 LC-9

205. Operation Condition + EQ(+X) (Serviceability): 0.8 LC-1+1.0 LC-3+0.8 LC-4+0.8 LC-5A+0.8 LC-5B+0.8 LC-5C+0.8 LC-5D+0.8 LC-7+0.8 LC-8+0.8 LC-9+0.8 LC-11A

206. Operation Condition + EQ (+Z) (Serviceability): 0.8 LC-2+1.0 LC-3+0.8 LC-4+0.8 LC-5A+0.8 LC-5B+0.8 LC-5C+0.8 LC-5D+0.8 LC-7+0.8 LC-8+0.8 LC-9+0.8 LC-11C

207. Operation Condition + EQ (-Z) (Serviceability): -0.8 LC-2+1.0 LC-3+0.8 LC-4+0.8 LC-5A+0.8 LC-5B+0.8 LC-5C+0.8 LC-5D+0.8 LC-7+0.8 LC-8+0.8 LC-9+0.8 LC-11B

208. Construction Condition + EQ (-X) (Serviceability): -0.8 LC-1+1.0 LC-3+0.8 LC-4+0.8 LC-6A+0.8 LC-6B+0.8 LC-6C+0.8 LC-9

209. Construction Condition + EQ (+Z) (Serviceability): 0.8 LC-2+1.0 LC-3+0.8 LC-4+0.8 LC-6A+0.8 LC-6B+0.8 LC-6C+0.8 LC-9+0.8 LC-12B

210. Construction Condition + EQ(-Z) (Serviceability): -0.8 LC-2+1.0 LC-3+0.8 LC-4+0.8 LC-6A+0.8 LC-6B+0.8 LC-6C+0.8 LC-9+0.8 LC-12A

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8 DESIGN CRITERIA

8.1 STRUCTURAL ANALYSIS

The Intake has been modelled in STAAD.pro corresponding to the centre line dimension. Material properties and loads have been applied accordingly. The results of Analysis have been used to design the section.

8.2 CLEAR COVER

Clear cover to reinforcement shall be as per EM-1100-2104, for Intake being in contact with water and atmosphere, the value of clear cover to reinforcement has been fixed as 75 mm for pier & wall and 25 mm for slab.

8.3 ULTIMATE LIMIT STATE

The Intake has been designed for strength for external forces such as Bending Moment (M) and shear force based on analysis result, for calculation refer **Annexure-II.**

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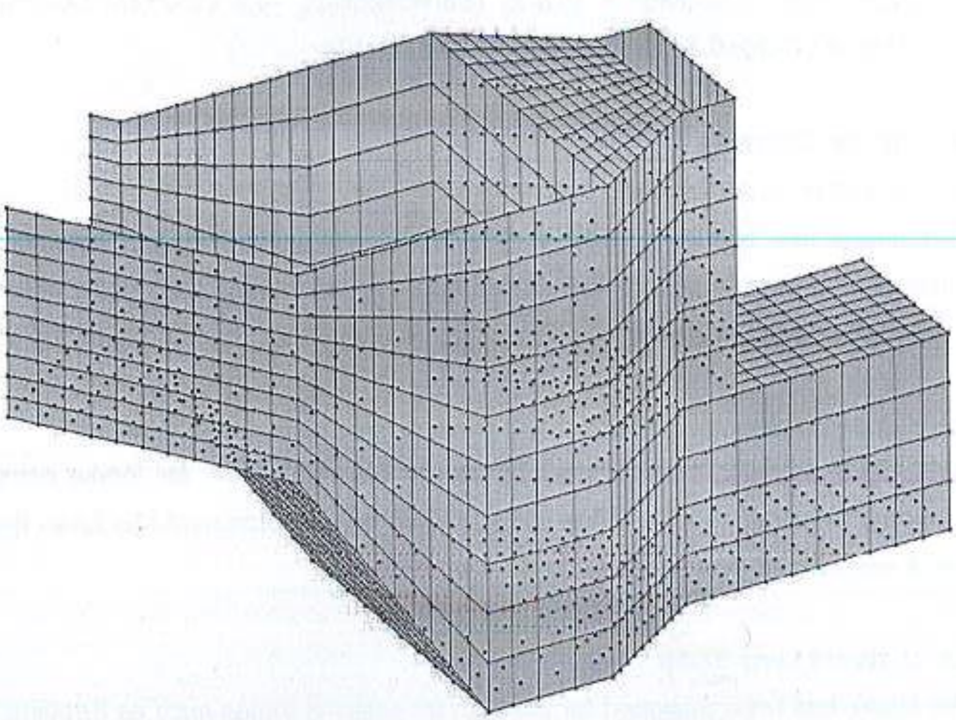

Figure- 2: Isometric View of Model

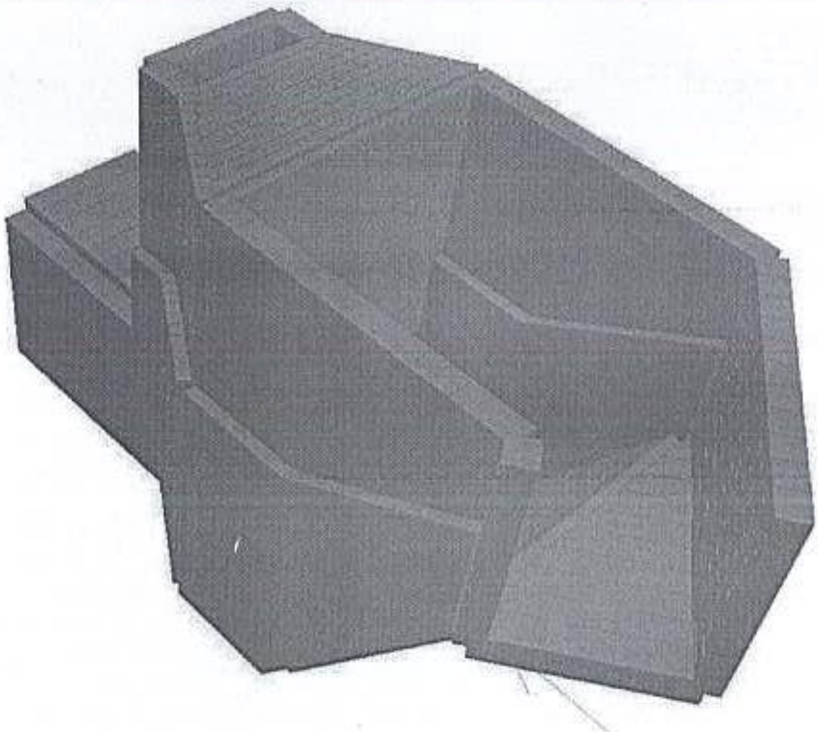

Figure- 3: 3D Render View of Model

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9 Result of Analysis and Design

9.1 Pier and Wall

S.No.	Component	Vertical Reinforcement		Horizontal Reinforcement	
		Diameter	Spacing	Diameter	Spacing
		(mm)	(mm)	(mm)	(mm)
1	Pier (Zone-1)	20	125	20	125
2	Pier (Zone 2A)	25	125	20	125
3	Pier (Zone 2B)	25	125	20	125
4	Breast Wall	20	150	25	150
5	Back Wall	20	150	20	150

9.2 Raft and Top Slab/Bridge

S.No.	Component	Across the Flow Reinforcement		Along the Flow Reinforcement	
		Diameter	Spacing	Diameter	Spacing
		(mm)	(mm)	(mm)	(mm)
1	Raft	25	125	20	150
2	Top Slab	16	200	16	200

Note: *For shear Reinforcement refer Annexure-II – F*

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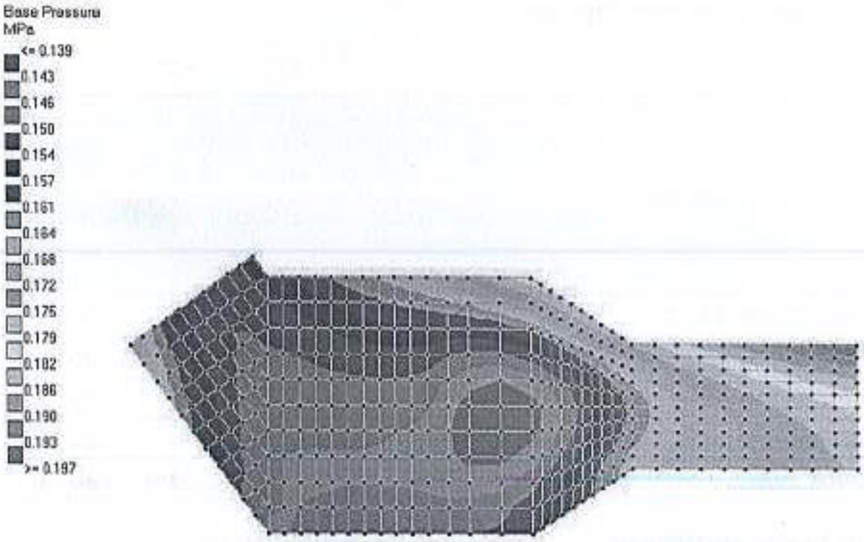

Figure- 4: Maximum Bearing pressure 197kN/m² < 200kN/m² -OK

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ANNEXURE-I

(LOAD CALCULATION)

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Khutani SHPP Project

Loads acting on the Intake structure:

Input:		
Top level of Intake	800	m
Maximum Water Level (MWL) =	799.00	m
Full Reservoir level (FRL) =	799.00	m
Top elevation of earth backfill =	800	m
Bottom elevation of earth backfill =	780.40	m
Top elevation of raft slab =	782.00	m
Bottom elevation of raft slab =	780.50	m
Bottom elevation of Breast wall =	788.50	m
Materials Properties:		
Grade of concrete =	25	N/mm^2
Unit weight of earthfill =	22	kN/m^3
Unit weight of water =	9.81	kN/m^3
Angle of internal friction of backfill soil (Φ) = (Assumed)	30	degrees
SIN Φ =	0.50	
Safe bearing capacity of soil =	200	kN/m^2
Coefficient of friction between foundation and soil (μ) =	0.58	
Ka	0.333	
Seismic Parameters:		
Horizontal seismic coefficient (αh) =	0.16	
Vertical seismic coefficient =	0.107	

Calculation of Loads:

1 **Dead Loads:**

a **Self wt. of the structure**

Calculated by software

2 **Live Load:**

a	**Point load (Due to hoisting arrangements) =**	160	kN
a	**Live Load Over Slab =**	5	kN/m2
c	**Internal Hydrostatic (MWL):**		
	Head =	17	m

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	Hydrostatic (MWL) =	166.77	kN/m²
d	External Hydrostatic (MWL):		
	Head =	0	m
	Water pressure =	0	kN/m²
e	Weight of water (MWL):		
	Head =	17	m
	Maximum Water pressure =	166.770	kN/m²
f	Uplift pressure (FRL):		
	Water head =	18.5	m
	Uplift pressure =	181.49	kN/m²
g	Water Pressure on Breast Wall:		
	Water head =	10.5	m
	Hydrostatic Pressure =	103.01	kN/m²
h	External soil Pressure on intake wall:		
	Height=	19.6	m
	Soil Pressure =	143.73	kN/m²
i	External soil Pressure on Back wall:		
	Height=	0	m
	Hydrostatic Pressure =	0.00	kN/m²
4	Seismic Loads:		
a	Seismic X	From Staad-Pro itself	
b	Seismic Z	From Staad-Pro itself	
c	Hydrodynamic X (FRL):	Refer Hydrodynamic pressure sheet	
d	Hydrodynamic Z (FRL):	Refer Hydrodynamic pressure sheet	
f	Hydrodynamic -Z (FRL):	Refer Hydrodynamic pressure sheet	

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Calculation of Hydrodynamic Pressure:

Input:
Design horizontal coefficient (αh) = **0.16**
Unit weight of water (w) = **9.81** kN/m³
Depth of reservoir (h) =

The Hydrodynamic Pressure

$$\boxed{p = C_s \alpha_h wh}$$

α_h Design Horizontal Seismic Coefficient
w unit weight of water kN/m³
h depth of reservoir,m

The Coefficient varies with shape and depth

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

C_m Coefficient Calculated from Fig:10 of IS:1893-1984
y Depth of water from surface of reservoir

1 Hydrodynamic pressure at +Z , _Z & -X-direction : **(Case-13, 14 & 15 in the STAAD.Pro)**

Top El.	=	**799**	m	
Bot. El.	=	**782**	m	
h	=	17	m	FRL
w	=	**9.81**	kN/m³	
c_m	=	**0.75**		
α_h	=	**0.16**		

Sl.No.	Element Height	Depth from Surface ,y	C_s	Hydrodynamic pressure
1	0	0	0.00	0.00
2	1.95	1.95	0.26	6.82
3	1.95	3.9	0.39	10.44
4	1.95	5.85	0.50	13.26
5	1.95	7.8	0.58	15.49
6	1.95	9.75	0.65	17.24
7	1.95	11.7	0.69	18.54
8	1.95	13.65	0.73	19.43
9	1.95	15.6	0.75	19.91
10	1.95	17.55	0.75	20.00
11	1.95	19.5	0.74	19.69

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Calculation of Dynamic Increment

1 **Input:**

As per IS:1893-1988 (part-1)

Top level of concrete =	**800.00**	m
Finished level of raft =	**782.50**	m
Bottom level of raft =	**780.00**	m
El. of earth reatined =	**800.00**	
Height of wall (Height of Earth to be retained)	17.50	m
Horizontal seismic coefficient, αh =	**0.16**	
Vertical seismic coefficient, αv =	0.11	
Dry Unit weight of soil =	**18**	kN/m^3
Sat. Unit weight of soil =	**22**	kN/m^3
Unit weight of water =	**9.81**	kN/m^3
Submerged unit weight, γ_{sub} =	12.19	kN/m^3
Angle of Internal friction, Φ =	**30**	degree
Angle which earth face of the wall makes with the vertical, α =	**0**	degree
Slope of earthfill, τ =	**0**	degree
Angle of friction between wall and earthfill, δ =	20.00	degree

$\alpha h/(1+\alpha v)$	=	0.145
$\alpha h/(1-\alpha v)$	=	0.179
$\lambda = \tan^{-1}(\alpha_h/1+\alpha_v)$	=	8.227
$\lambda = \tan^{-1}(\alpha_h/1-\alpha_v)$	=	10.154

Dynamic Earth Pressure:

The pressure from earth fill behind concrete lining during an earthquake shall be as given in IS 1893-1984, Clause 8.1.1 to 8.1.4. In the analysis, cohesion has been neglected.

Active Pressure due to Earth fill under Seismic condition is given as,

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$$Pa = \frac{1}{2} Ca * \gamma * H^2$$

2 Calculation of Dynamic Coefficient (Ca):

		When αv,αh and λ =0,	
angle(Φ-λ-α)	21.77	30.00	degree
angle(δ+α+λ)	28.23	20.00	degree
angle(Φ+δ)	50.00	50.00	degree
angle(Φ-τ-λ)	21.77	30.00	degree
angle(α-τ)	0.00	0.00	degree
$\cos^2(\Phi-\lambda-\alpha)$	0.86	0.75	
$\cos(\delta+\alpha+\lambda)$	0.88	0.94	
$\sin(\Phi+\delta)$	0.77	0.77	
$\sin(\Phi-\tau-\lambda)$	0.37	0.50	
$\cos(\alpha-\tau)$	1.00	1.00	
$\cos^2\alpha$	1.000	1.000	
$\cos\lambda$	0.9897	1.0000	
$1+\alpha_v$	1.107	1.11	
$1-\alpha_v$	0.893	0.89	

$$C_a = \frac{(1+\alpha_v)*(\cos^2(\Phi-\lambda-\alpha))}{(\cos\lambda)*(\cos^2(\alpha))*(\cos(\delta+\alpha+\lambda))} * \left(\frac{1}{1+\left(\frac{\sin(\Phi+\delta)*(\sin(\Phi-\tau-\lambda))}{\cos(\alpha-\tau)*\cos(\delta+\alpha+\lambda)}\right)^{1/2}}\right)^2$$

(i) *Coefficient of Dynamic Increment, Ca =* **0.445** **0.329**

$$C_a = \frac{(1-\alpha_v)*(\cos^2(\Phi-\lambda-\alpha))}{(\cos\lambda)*(\cos^2(\alpha))*(\cos(\delta+\alpha+\lambda))} * \left(\frac{1}{1+\left(\frac{\sin(\Phi+\delta)*(\sin(\Phi-\tau-\lambda))}{\cos(\alpha-\tau)*\cos(\delta+\alpha+\lambda)}\right)^{1/2}}\right)^2$$

(ii) *Coefficient of Dynamic Increment, Ca =* **0.359**

Considering Higher value of (i) and (ii), Ca = 0.445

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For α_v,α_h and λ =0 ,we get Ca as 0.329

0.116

Total Pressure at base, pa (Dynamic) =	***266.65***	***kN/m²***	***Dynamic Earth Pressure for Ca =***	0.445

$$\boxed{p_a(Submerged) = C_a * \gamma_{sub} * H + \gamma_w * H}$$

Total Pressure Intensity, Pa =	***2333.19***	***kN/m***

$$\boxed{Pa = \frac{1}{2} Ca\ \gamma H^2}$$

Total Pressure at base, pa (Static) =	***241.86***	***kN/m²***	***Static Earth Pressure for Ca =***	0.329
Total Pressure Intensity, Pa =	***2116.32***	***kN/m***		
Dynamic Increment =	***216.88***	***kN/m***	***24.79***	***kN/m²***

3 **Point of application:** (Clause 8.1.1.2, IS: 1893-1984)

Dynamic Increment acts at H/2 from Base i.e. 8.75 m

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ANNEXURE-II

(STRUCTURAL DESIGN-REINFORCEMENT CALCULATION OF INTAKE COMPONENT)

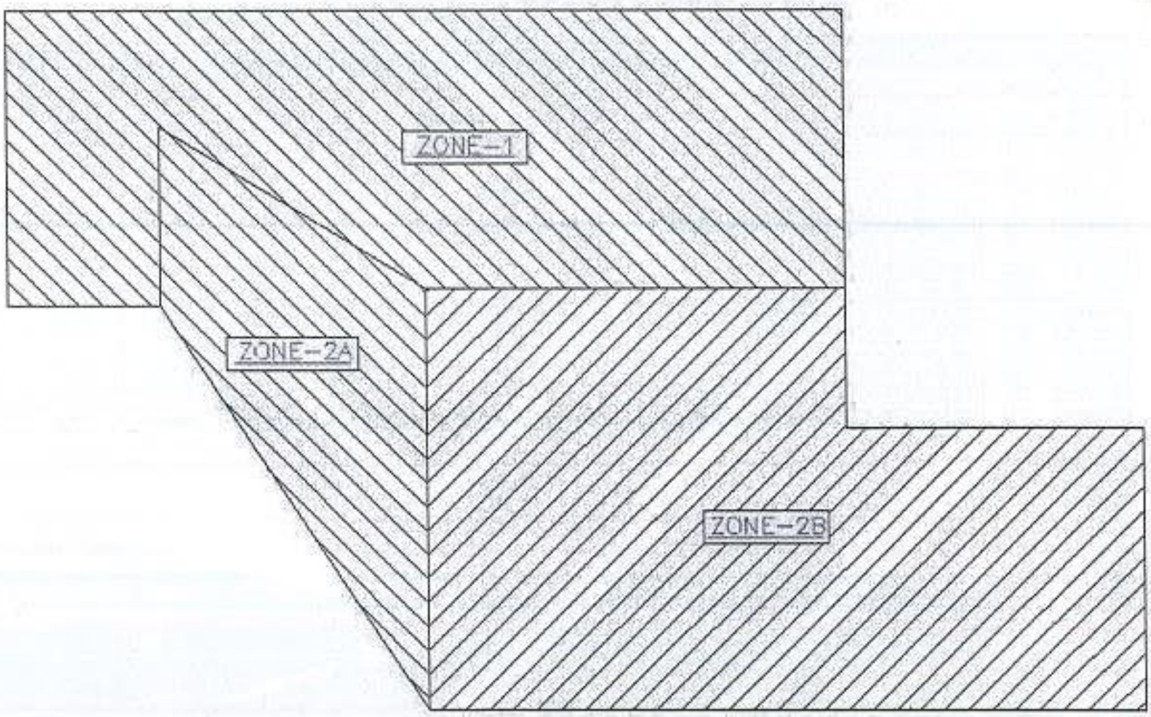

Figure- 5: Pier Zoning for Reinforcement Calculations

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A. BENDING MOMENT FOR PIER FOR ZONE-1 IN X & Y DIRECTION:

PIER ZONE-1 (STRESS SUMMARY)											
			Shear (Local)		Membrane (Local)			Bending Moment (Local)			
	Plate	L/C	SQX MPa	SQY MPa	SX MPa	SY MPa	SXY MPa	MX kN-m/m	MY kN-m/m	MXY kN-m/m	
Max Qx	216	109 CONSTRUCTION CONDITION + EQ (+Z)	0.482	-0.068	0.169	0.01	-0.084	413.516	48.235	134.5	
Min Qx	2536	106 OPERATION CONDITION + EQ (+Z)	-0.603	0.286	0.714	-0.574	-0.947	495.724	157.579	-52.47	
Max Qy	2436	110 CONSTRUCTION CONDITION + EQ (-Z)	0.03	0.437	-0.007	-0.616	0.076	-40.898	-729.89	-56.935	
Min Qy	2436	106 OPERATION CONDITION + EQ (+Z)	-0.016	-0.4	-0.007	0.105	-0.058	29.16	445.111	77.503	
Max Sx	243	105 OPERATION CONDITION + EQ(+X)	-0.077	-0.041	1.26	-0.439	-0.302	78.869	12.171	-59.101	
Min Sx	409	110 CONSTRUCTION CONDITION + EQ (-Z)	0.31	0.006	-0.994	-0.002	0.014	-200.538	-10.704	57.899	
Max Sy	128	105 OPERATION CONDITION + EQ(+X)	-0.023	-0.059	0.175	0.597	-0.656	28.739	43.725	-51.7	
Min Sy	2547	107 OPERATION CONDITION + EQ (-Z)	0.034	0.1	0.07	-0.854	-0.007	-37.197	-48.578	-12.857	
Max Sxy	2526	110 CONSTRUCTION CONDITION + EQ (-Z)	0.028	0.016	0.092	-0.125	0.656	-197.766	-217.251	18.546	
Min Sxy	2536	106 OPERATION CONDITION + EQ (+Z)	-0.603	0.286	0.714	-0.574	-0.947	495.724	157.579	-52.47	
Max Mx	2539	106 OPERATION CONDITION + EQ (+Z)	-0.325	0.059	0.481	-0.333	-0.454	632	147.25	29.471	
Min Mx	407	110 CONSTRUCTION CONDITION + EQ (-Z)	0.421	-0.055	-0.934	-0.009	0.052	-687	-29.165	19.087	
Max My	2436	106 OPERATION CONDITION + EQ (+Z)	-0.016	-0.4	-0.007	0.105	-0.058	29.16	445	77.503	
Min My	2436	110 CONSTRUCTION CONDITION + EQ (-Z)	0.03	0.437	-0.007	-0.616	0.076	-40.898	-730	-56.935	
Max Mxy	116	109 CONSTRUCTION CONDITION + EQ (+Z)	-0.027	0.034	0.022	-0.035	-0.043	-266.616	-92.321	237.429	
Min Mxy	2513	110 CONSTRUCTION CONDITION + EQ (-Z)	0.1	-0.002	-0.374	-0.044	0.198	-100.647	-2.851	-162.718	

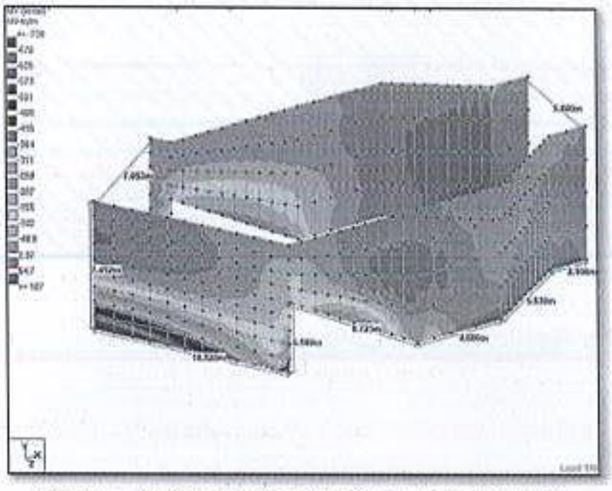

Figure- 6: Maximum Bending Moment (My) Pier- Zone-1

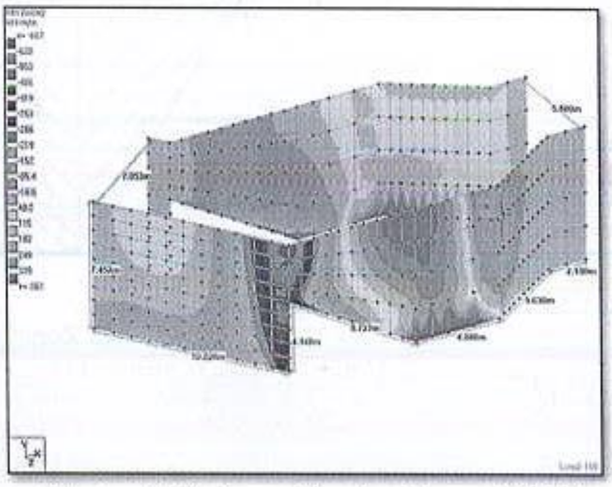

Figure- 7: Maximum Bending Moment (Mx) Pier- Zone-1

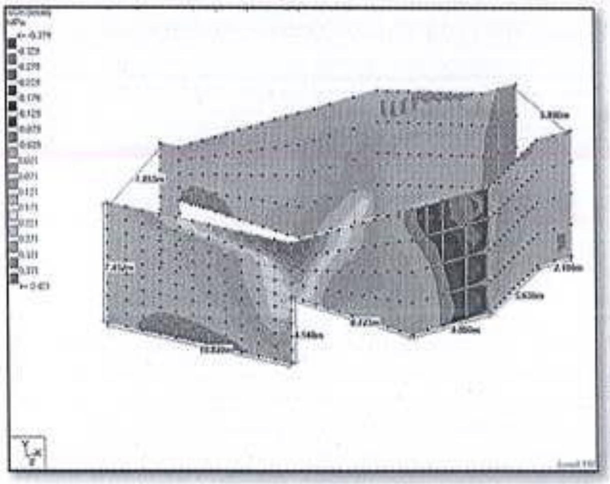

Figure- 8: Shear Force (S_QY) in Pier- Zone-1

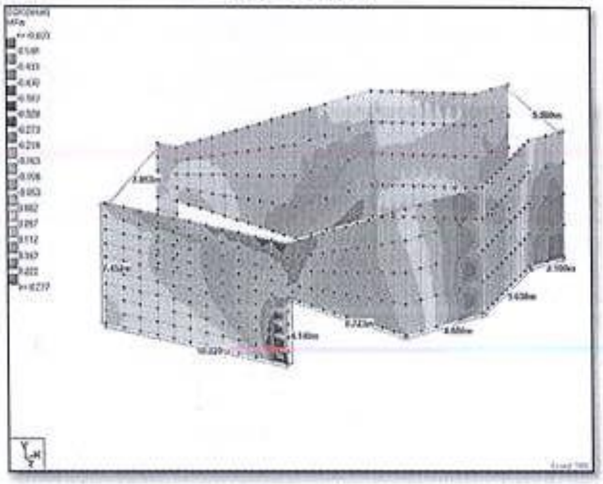

Figure- 9: Shear Force (S_QX) in Pier- Zone-1

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DESIGN OF PIER (ZONE-1) FOR BENDING MOMENT IN Y DIRECTION (VERTICAL):

Input from STAAD FILE		
Maximum Bending moment in Y direction	730	KN-m/m
Bending moment in maximum area of pier	730	KN-m/m
Design		
Fy	500	N/mm^2
Fck	25	N/mm^2
Clear Cover	75	mm
Diameter of bar	20	mm
Width	1000	mm
Thickness of pier	1000	mm
Effective depth	915	mm
Design Moment Mu	730	KN-m/m
Mu/b*d^2	0.87	
Pt Required $Pt=\left(\frac{Fck}{2\times Fy}\right)(1-\sqrt{1}-4.598\times R/Fck)$	0.21	%
Ast Required	1914	OK
Spacing required using φ20	164	mm
Spacing Provided	125	mm
Ast Provided	2513	mm
Minimum reinforcement @ 0.12% of cross section area	1098	mm2
	2513	
Provided vertical reinforcement φ20 @ 125 mm c/c spacing		

DESIGN OF PIER (ZONE-1) FOR BENDING MOMENT IN X DIRECTION (HORIZONTAL):

Input from STAAD FILE		
Maximum Bending moment in X direction	687	KN-m/m
Bending moment in maximum area of pier	687	KN-m/m
Design		
Fy	500	N/mm^2
Fck	25	N/mm^2
Clear Cover	75	mm
Diameter of bar	20	mm
Width	1000	mm
Thickness of pier	1000	mm
Effective depth	915	mm
Design Moment Mu	687	KN-m/m
Mu/b*d^2	0.82	
Pt Required $Pt=\left(\frac{Fck}{2\times Fy}\right)(1-\sqrt{1}-4.598\times R/Fck)$	0.20	
Ast Required	1796	OK
Spacing required using φ20	175	mm
Minimum reinforcement @ 0.12% of cross section area	1098	
Dia of bar	25	mm
Spacing	125	mm
No of bar in 1 m	8	nos.
Actual Ast provided	3925	mm2
	OK	
Provided Horizontal reinforcement φ20 @ 125 mm c/c spacing		

B. Bending Moment for Pier for Zone-2A in X & Y Direction:

PIER ZONE-2A (STRESS SUMMARY)										
			Shear (Local)		Membrane (Local)		Bending Moment (Local)			
	Plate	L/C	SQX	SQY	SX MPa	SY MPa	SXY	MX kN-	MY kN-	MXY kN-m/m
Max Qx	165	107 OPERATION CONDITION + EQ (-Z)	0.805	-0.434	1.454	1.035	-1.469	-1263	-2876.5	57.003
Min Qx	182	109 CONSTRUCTION CONDITION + EQ (+Z)	-0.924	1.268	0.201	0.119	-0.251	2127.2	4028.35	368.726
Max Qy	182	109 CONSTRUCTION CONDITION + EQ (+Z)	-0.924	1.268	0.201	0.119	-0.251	2127.2	4028.35	368.726
Min Qy	182	107 OPERATION CONDITION + EQ (-Z)	0.784	-1.107	1.376	2.17	-1.89	-1713.4	-3482.9	-125.243
Max Sx	165	105 OPERATION CONDITION + EQ(+X)	0.253	-0.165	1.469	1.226	-1.321	-349	-647.59	-21.101
Min Sx	487	106 OPERATION CONDITION + EQ (+Z)	-0.083	0.177	-0.645	1.077	-0.701	443.462	1107.65	-171.047
Max Sy	182	105 OPERATION CONDITION + EQ(+X)	0.238	-0.389	1.437	2.355	-1.72	-508.06	-905.6	-101.403
Min Sy	496	109 CONSTRUCTION CONDITION + EQ (+Z)	-0.123	-0.025	0.795	-0.897	-0.564	55.762	736.91	-40.844
Max Sxy	457	110 CONSTRUCTION CONDITION + EQ (-Z)	-0.029	-0.084	0.128	-0.08	0.254	-379.28	-225.15	296.865
Min Sxy	182	107 OPERATION CONDITION + EQ (-Z)	0.784	-1.107	1.376	2.17	-1.89	-1713.4	-3482.9	-125.243
Max Mx	182	109 CONSTRUCTION CONDITION + EQ (+Z)	-0.924	1.268	0.201	0.119	-0.251	2127	4028.35	368.726
Min Mx	182	107 OPERATION CONDITION + EQ (-Z)	0.784	-1.107	1.376	2.17	-1.89	-1713	-3482.9	-125.243
Max My	182	109 CONSTRUCTION CONDITION + EQ (+Z)	-0.924	1.268	0.201	0.119	-0.251	2127.2	4028	368.726
Min My	182	107 OPERATION CONDITION + EQ (-Z)	0.784	-1.107	1.376	2.17	-1.89	-1713.4	-3483	-125.243
Max Mxy	185	109 CONSTRUCTION CONDITION + EQ (+Z)	0.117	0.647	-0.026	-0.105	-0.14	1599.28	2019.21	878.812
Min Mxy	185	107 OPERATION CONDITION + EQ (-Z)	-0.136	-0.518	0.053	1.582	-0.599	-1193	-1615.2	-618.668

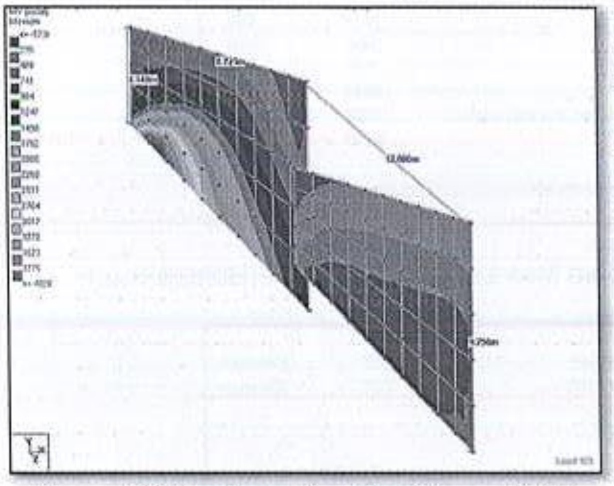

Figure- 10: Maximum Bending Moment (My) Pier- Zone-2A

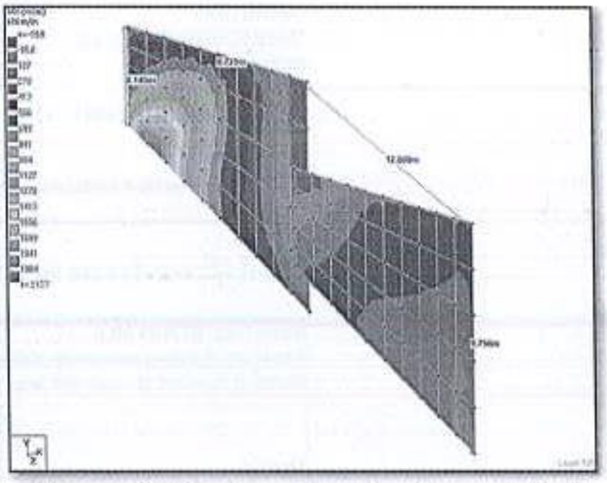

Figure- 11: Maximum Bending Moment (Mx) Pier- Zone-2A

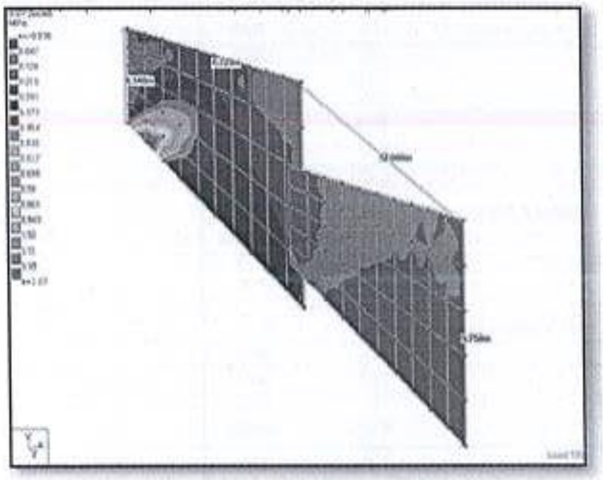

Figure- 12: Shear Force (S_QY) in Pier- Zone-2A

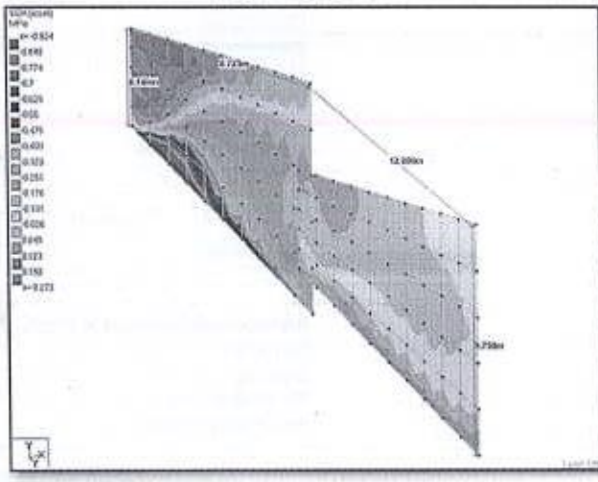

Figure- 13: Shear Force (S_QX) in Pier- Zone-2A

DESIGN OF PIER (ZONE-2A) FOR BENDING MOMENT IN Y DIRECTION (VERTICAL):

Input from STAAD FILE		
Maximum Bending moment in Y direction	4028	KN-m/m
Bending moment in maximum area of pier	3100	KN-m/m
Design		
Fy	500	N/mm^2
Fck	25	N/mm^2
Clear Cover	75	mm
Diameter of bar	25	mm
Width	1000	mm
Thickness of pier	2000	mm
Effective depth	1912.5	mm
Design Moment Mu	3100	KN-m/m
Mu/b*d^2	0.85	
Pt Required $Pt = \left(\frac{Fck}{2\times Fy}\right)\left(1-\sqrt{1-4.598\times R/Fck}\right)$	0.20	
Ast Required	3884	OK
Layer -1		
Diameter of bar	25	mm
Spacing of bar	125	mm
Provided Reinforcement	3927	mm2
Minimum reinforcement @ 0.12% of cross section area	2295	mm2
Provided vertical reinforcement φ25 @ 125 mm c/c spacing		

DESIGN OF PIER (ZONE-2A) FOR BENDING MOMENT IN X DIRECTION (HORIZONTAL):

Input from STAAD FILE		
Maximum Bending moment in X direction	2127.2	KN-m/m
Bending moment in maximum area of pier	2000.0	KN-m/m
Maximum bending moment in X direction is occurred at point on the corner, for resist this moment extra oblique bar is provided at the corner, however pier is designed for average maximum bending moment in pier i.e 2000 KN-m/m		
Design		
Fy	500	N/mm^2
Fck	25	N/mm^2
Clear Cover	75	mm
Diameter of bar	20	mm
Width	1000	mm
Thickness of pier	2000	mm
Effective depth	1915	mm
Design Moment Mu	2000	KN-m/m
Mu/b*d^2	0.55	
Pt Required $Pt = \left(\frac{Fck}{2\times Fy}\right)\left(1-\sqrt{1-4.598\times R/Fck}\right)$	0.13	
Total Ast Required	2464	OK
No of Layer	2	
Spacing required using φ20	127	mm
Minimum reinforcement @ 0.12% of cross section area	2298	
Dia of bar	20	mm
Spacing	125	mm
No of bar in 1 m	8	nos.
Actual Ast provided	2512	mm2
	OK	
Provided Horizontal reinforcement φ20 @ 125 mm c/c spacing		

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C. Bending Moment (Zone-2B) FOR Pier at Top Portion in X & Y Direction:

PIER ZONE-2B (STRESS SUMMARY)										
			Shear (Local)		Membrane (Local)		Bending Moment (Local)			
	Plate	L/C	SQX	SQY	SX MPa	SY MPa	SXY	MX kN-	MY kN-	MXY kN-
Max Qx	566	107 OPERATION CONDITION + EQ (-Z)	0.48	-0.013	0.48	-0.248	-0.066	1342.71	545.612	60.065
Min Qx	266	106 OPERATION CONDITION + EQ (+Z)	-0.477	0.015	0.475	-0.304	-0.079	-1306.2	-570.87	-84.075
Max Qy	1179	106 OPERATION CONDITION + EQ (+Z)	0.195	0.354	0.453	-0.246	-0.391	-1087.9	-419.63	-80.561
Min Qy	979	107 OPERATION CONDITION + EQ (-Z)	-0.2	-0.367	0.453	-0.186	-0.358	1102.89	397.68	57.725
Max Sx	253	105 OPERATION CONDITION + EQ(+X)	-0.096	-0.027	1.317	-0.26	0.029	463.343	-91.396	-231.665
Min Sx	1299	102 OPERATION CONDITION 2	-0.08	0.026	-0.255	-0.549	0.004	40.421	072.399	-132.029
Max Sy	996	107 OPERATION CONDITION + EQ (-Z)	-0.158	-0.031	0.516	0.156	-0.03	360.107	-768.93	147.699
Min Sy	1207	101 OPERATION CONDITION-1	-0.163	0.074	-0.065	-0.946	0.229	312.979	859.702	-17.131
Max Sxy	293	107 OPERATION CONDITION + EQ (-Z)	0.422	0.018	0.749	-0.541	1.076	-163.38	-274.24	599.663
Min Sxy	1163	106 OPERATION CONDITION + EQ (+Z)	0.167	0.195	0.329	-0.327	-0.439	-177.17	-230.84	36.908
Max Mx	556	107 OPERATION CONDITION + EQ (-Z)	0.322	0.058	0.538	-0.16	-0.074	1650	239.915	-79.457
Min Mx	256	106 OPERATION CONDITION + EQ (+Z)	-0.325	-0.058	0.516	-0.202	-0.083	-1602	-252.24	55.483
Max My	1203	102 OPERATION CONDITION-2	0.013	0.052	0.182	-0.565	0.254	493.409	994	4.369
Min My	1003	102 OPERATION CONDITION-2	-0.013	-0.052	0.186	-0.528	0.234	-470.73	-1026	-19.009
Max Mxy	293	107 OPERATION CONDITION + EQ (-Z)	0.422	0.018	0.749	-0.541	1.076	-163.38	-274.24	599.663
Min Mxy	593	106 OPERATION CONDITION + EQ (+Z)	-0.435	-0.006	0.668	-0.592	0.954	173.142	243.404	-613.004

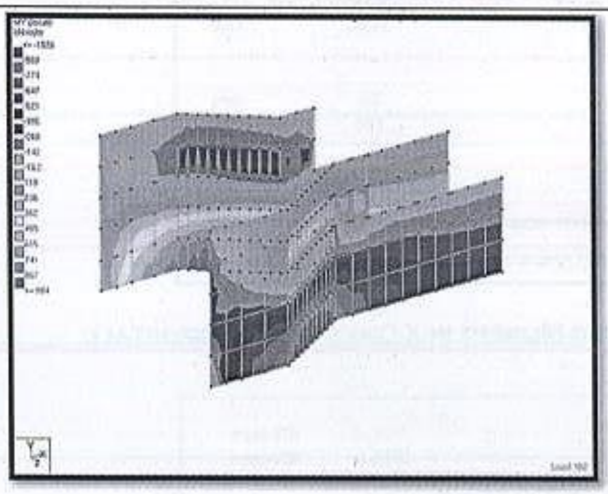

Figure- 14: Maximum Bending Moment (My) Pier- Zone-2B

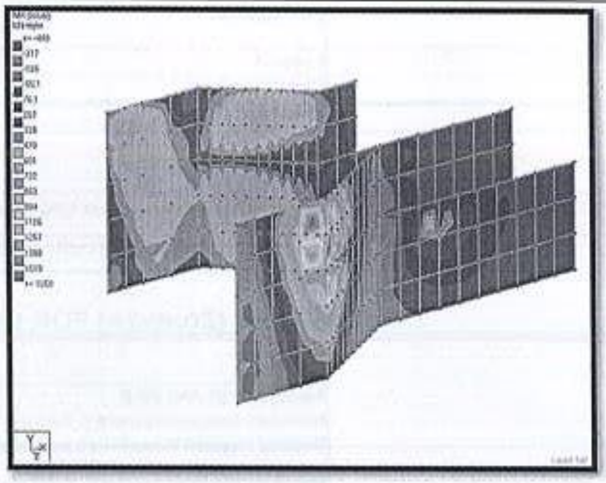

Figure- 15: Maximum Bending Moment (Mx) Pier- Zone-2B

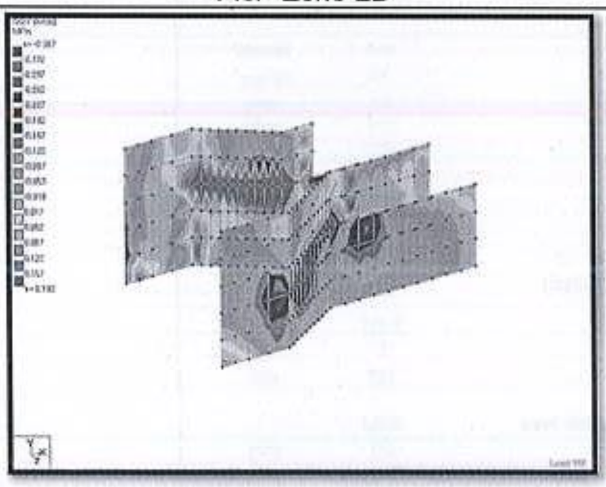

Figure- 16: Shear Force (S_QY) in Pier- Zone-2B

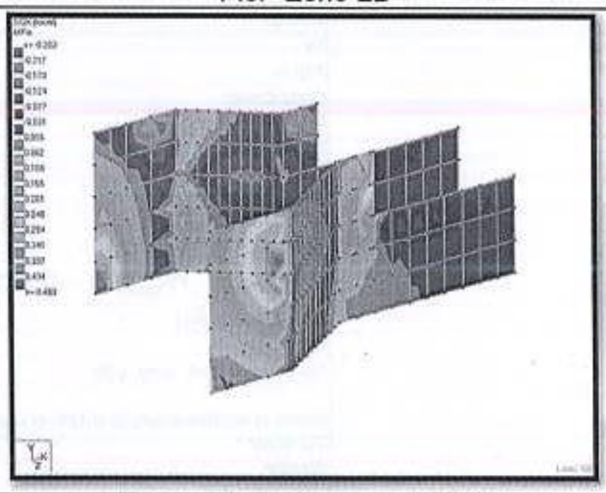

Figure- 17: Shear Force (S_QX) in Pier- Zone-2B

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DESIGN OF PIER (ZONE-2B) FOR BENDING MOMENT IN Y DIRECTION (VERTICAL):

Input from STAAD FILE		
Maximum Bending moment in Y direction	1026	KN-m/m
Bending moment in maximum area of pier	1026	KN-m/m
Maximum bending moment in Y direction is occurred at point on the corner, for resist this moment extra oblique bar is provided at the corner, however pier is designed for average maximum bending moment in pier i.e 998 KN-m/m		
Design		
Fy	500	N/mm^2
Fck	25	N/mm^2
Clear Cover	75	mm
Diameter of bar	25	mm
Width	1000	mm
Thickness of pier	2000	mm
Effective depth	1912.5	mm
Design Moment Mu	1026	KN-m/m
Mu/b*d^2	0.28	
Pt Required $Pt=\left(\frac{Fck}{2\times Fy}\right)\left(1-\sqrt{1-4.598\times R/Fck}\right)$	0.07	
Ast Required	1250	NOT OK
Spacing required using 25 dia bars	125	mm
Minimum reinforcement @ 0.12% of cross section area	2295	mm2
Provided vertical reinforcement φ25 @ 125 mm c/c spacing		

DESIGN OF PIER (ZONE-2B) FOR BENDING MOMENT IN X DIRECTION (HORIZONTAL):

Input from STAAD FILE		
Maximum Bending moment in X direction	1650.3	KN-m/m
Bending moment in maximum area of pier	1650.3	KN-m/m
Maximum bending moment in X direction is occurred at point on the corner, for resist this moment extra oblique bar is provided at the corner, however pier is designed for average maximum bending moment in pier i.e 1630.2 KN-m/m		
Design		
Fy	500	N/mm^2
Fck	25	N/mm^2
Clear Cover	75	mm
Diameter of bar	20	mm
Width	1000	mm
Thickness of pier	2000	mm
Effective depth	1915	mm
Design Moment Mu	1650	KN-m/m
Mu/b*d^2	0.45	
Pt Required $Pt=\left(\frac{Fck}{2\times Fy}\right)\left(1-\sqrt{1-4.598\times R/Fck}\right)$	0.11	
Total Ast Required	2024	NOT OK
No of Layer	1	
Spacing required using φ20	155	mm
Minimum reinforcement @ 0.12% of cross section area	2298	
Dia of bar	20	mm
Spacing	125	mm
No of bar in 1 m	8	nos.
Actual Ast provided	2512	mm2
	OK	
Provided Horizontal reinforcement φ20 @ 125 mm c/c spacing		

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D. BENDING MOMENT FOR RAFT IN X & Y DIRECTION:

			Shear (Local)		Membrane (Local)			Bending Moment (Local)		
	Plate	L/C	SQX MPa	SQY MPa	SX MPa	SY MPa	SXY MPa	MX kN-m/m	MY kN-m/m	MXY kN-m/m
Max Qx	769	102 OPERATION CONDITION-2	0.411	-0.019	-1.056	-0.147	0.242	333.662	-29.653	-11.698
Min Qx	866	102 OPERATION CONDITION-2	-0.611	-0.025	0	0	0	358.094	-153.778	-34.996
Max Qy	1106	102 OPERATION CONDITION-2	-0.01	0.693	0	0	0	118.97	194.545	165.462
Min Qy	1036	106 OPERATION CONDITION + EQ (+Z)	-0.038	-0.7	0	0	0	186.007	539.822	-45.685
Max Sx	722	107 OPERATION CONDITION + EQ (-Z)	0.001	0.034	0.939	0.148	-1.141	-8.345	-60.457	21.756
Min Sx	769	102 OPERATION CONDITION-2	0.411	-0.019	-1.056	-0.147	0.242	333.662	-29.653	-11.698
Max Sy	721	105 OPERATION CONDITION + EQ(+X)	-0.003	0.017	0.839	0.159	-0.259	-11.774	-26.754	27.637
Min Sy	769	102 OPERATION CONDITION-2	0.411	-0.019	-1.056	-0.147	0.242	333.662	-29.653	-11.698
Max Sxy	799	106 OPERATION CONDITION + EQ (+Z)	-0.05	-0.034	0.164	0.044	2.387	-27.537	-42.929	-72.697
Min Sxy	729	107 OPERATION CONDITION + EQ (-Z)	-0.077	0.032	0.253	0.058	-2.326	-46.043	-53.209	74.365
Max Mx	2411	109 CONSTRUCTION CONDITION + EQ (+Z)	0.258	0.619	0	0	0	571	521.329	65.501
Min Mx	869	102 OPERATION CONDITION-2	-0.042	0.052	0	0	0	-575	-655.88	-34.274
Max My	2368	110 CONSTRUCTION CONDITION + EQ (-Z)	-0.014	-0.551	0	0	0	45.853	581	85.727
Min My	1071	102 OPERATION CONDITION-2	0.129	0.076	0	0	0	-348.303	-754	-10.509
Max Mxy	2421	107 OPERATION CONDITION + EQ (-Z)	-0.013	-0.42	0	0	0	-147.305	-168.606	248.707
Min Mxy	2416	106 OPERATION CONDITION + EQ (+Z)	-0.109	0.23	0	0	0	209.181	347.924	-256.383

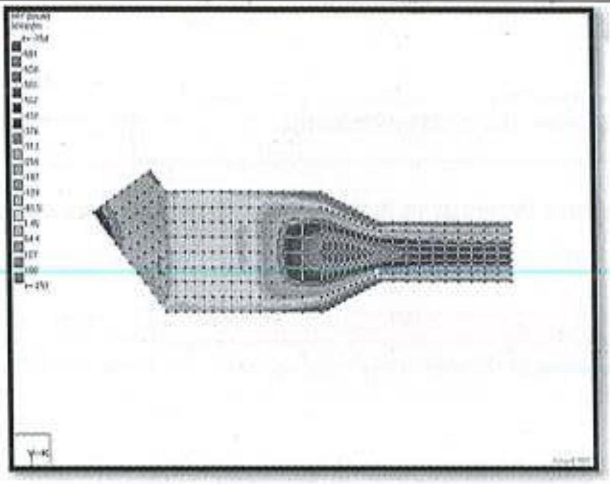

Figure- 18: Maximum Bending Moment (My) Raft

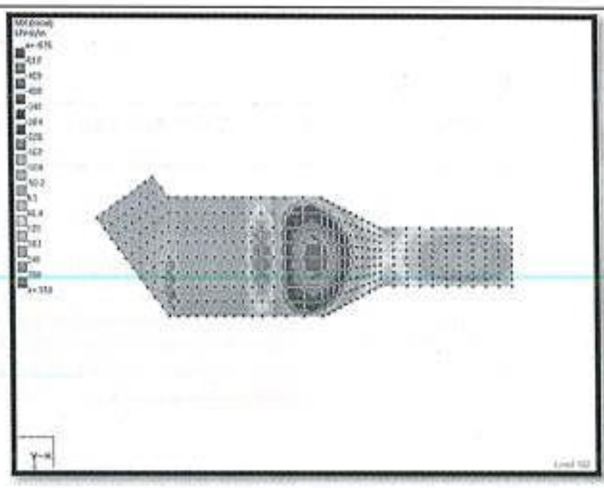

Figure- 19: Maximum Bending Moment (Mx) Raft

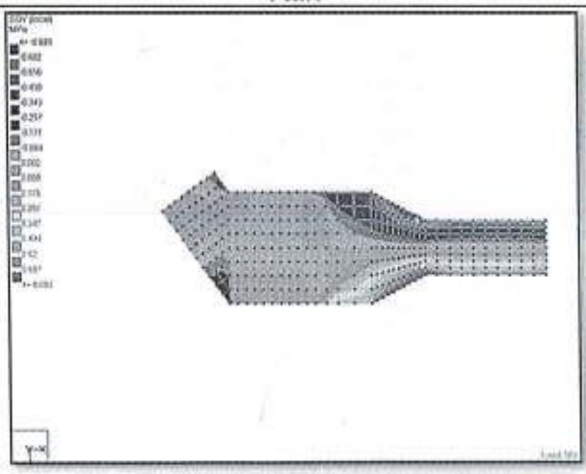

Figure- 20: Shear Force in Raft (SQY)

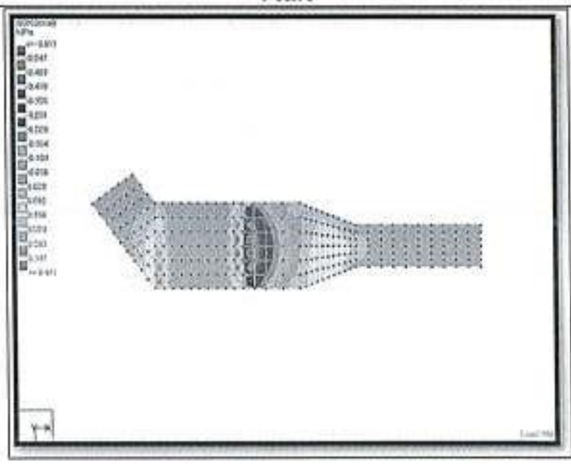

Figure- 21: Shear Force in Raft (SQX)

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Design of Raft for Bending Moment in Y Direction (Across the Flow):

Input from STAAD FILE			
Maximum Bending moment in Y direction		754	KN-m/m
Design			
Fy		500	N/mm^2
Fck		25	N/mm^2
Clear Cover		75	mm
Diameter of bar		25	mm
Width		1000	mm
Thickness of raft		1200	mm
Effective depth		1112.5	mm
Design Moment Mu		754	KN-m/m
Mu/b*d^2		0.61	
Pt Required	$Pt=\left(\frac{Fck}{2\times Fy}\right)\left(1-\sqrt{1-4.598\times R/Fck}\right)$	0.14	
Ast Required		1604	**OK**
Spacing required using 25 dia bars		306	mm
Spacing Provided		125	mm
Minimum reinforcement @ 0.12% of cross section area		1335	mm2
Provided across the flow reinforcement φ25 @ 125 mm c/c spacing			

Design of Raft for Bending Moment in X Direction (Along the Flow):

Input from STAAD FILE			
Maximum Bending moment in X direction		575	KN-m/m
Design			
Fy		500	N/mm^2
Fck		25	N/mm^2
Clear Cover		75	mm
Diameter of bar		20	mm
Width		1000	mm
Thickness of raft		1200	mm
Effective depth		1115	mm
Design Moment Mu		575	KN-m/m
Mu/b*d^2		0.46	
Pt required	$Pt=\left(\frac{Fck}{2\times Fy}\right)\left(1-\sqrt{1-4.598\times R/Fck}\right)$	0.11	
Ast required		1212	**NOT OK**
Spacing required using 20 dia bars		259	mm
Minimum reinforcement @ 0.12% of cross section area		1338	mm2
Dia		20	mm
Spacing		150	mm
No of bar in 1 m		6.67	nos.
Actual Ast provided		2093	mm2
		OK	
Provided along the flow reinforcement φ20 @ 150 mm c/c spacing			

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E. BENDING MOMENT FOR BREAST WALL IN X & Y DIRECTION:

			Shear (Local)	Bending Moment (Local)			
	Plate	L/C	SQX MPa	SQY	MX kN-	MY kN-	MXY kN-
Max Qx	2061	105 OPERATION CONDITION + EQ(+X)	0.361	0.007	-66.688	57.955	-30.08
Min Qx	2054	105 OPERATION CONDITION + EQ(+X)	-0.361	0.003	-68.063	57.522	26.847
Max Qy	1739	107 OPERATION CONDITION + EQ (-Z)	0.029	0.697	-105.303	-352.391	-69.936
Min Qy	1809	106 OPERATION CONDITION + EQ (+Z)	-0.048	-0.706	-95.435	-375.443	43.711
Max Sx	2074	106 OPERATION CONDITION + EQ (+Z)	-0.041	0.051	-69.735	-49.353	92.583
Min Sx	2081	109 CONSTRUCTION CONDITION + EQ (+Z)	-0.049	0.071	46.302	-26.165	-1.41
Max Sy	1780	102 OPERATION CONDITION-2	-0.016	-0.032	3.007	172.333	3.193
Min Sy	1801	109 CONSTRUCTION CONDITION + EQ (+Z)	-0.155	-0.206	28.262	-38.868	28.031
Max Sxy	1731	106 OPERATION CONDITION + EQ (+Z)	0.011	-0.233	20.518	143.344	-73.972
Min Sxy	1808	107 OPERATION CONDITION + EQ (-Z)	-0.079	-0.333	24.749	79.379	25.915
Max Mx	1771	105 OPERATION CONDITION + EQ(+X)	0.089	0.066	454	164.48	32.335
Min Mx	1739	107 OPERATION CONDITION + EQ (-Z)	0.029	0.697	-105	-352.391	-69.936
Max My	1780	101 OPERATION CONDITION-1	-0.039	-0.095	9.174	425	11.546
Min My	1809	106 OPERATION CONDITION + EQ (+Z)	-0.048	-0.706	-95.435	-375	43.711
Max Mxy	1792	101 OPERATION CONDITION-1	-0.009	-0.107	225.091	171.85	200.423
Min Mxy	1742	101 OPERATION CONDITION-1	-0.004	0.115	175.692	218.298	-200.417

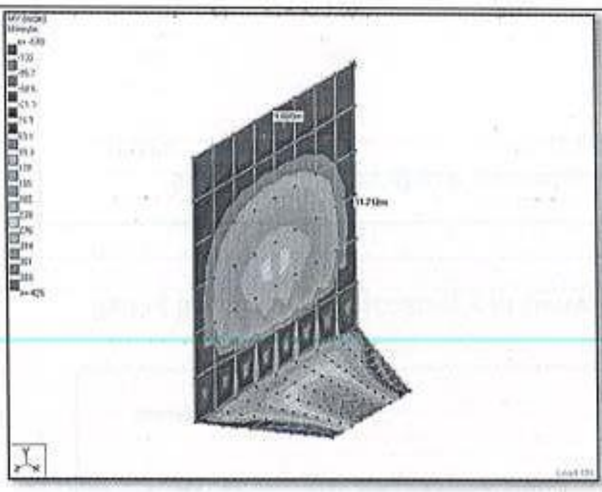

Figure- 22: Maximum Bending Moment (My) Breast Wall

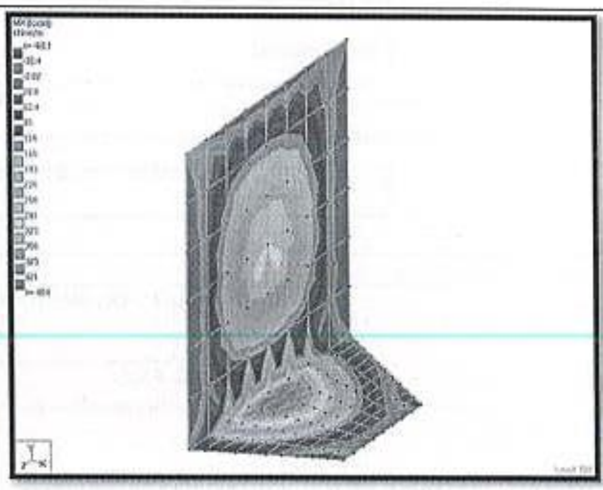

Figure- 23: Maximum Bending Moment (Mx) Breast Wall

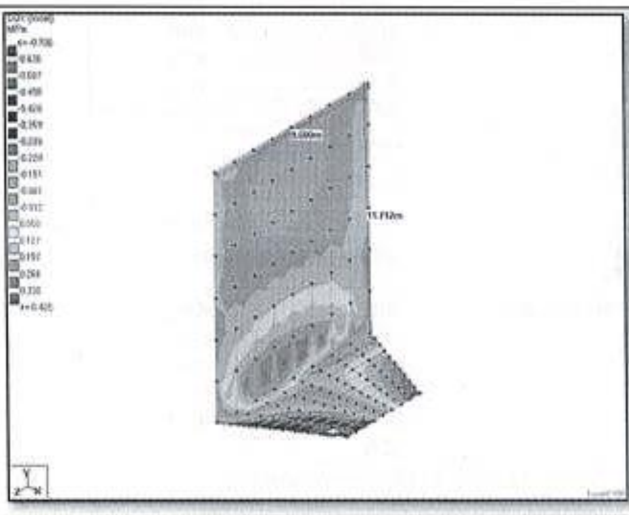

Figure- 24: Shear Force in Breast Wall (SQY)

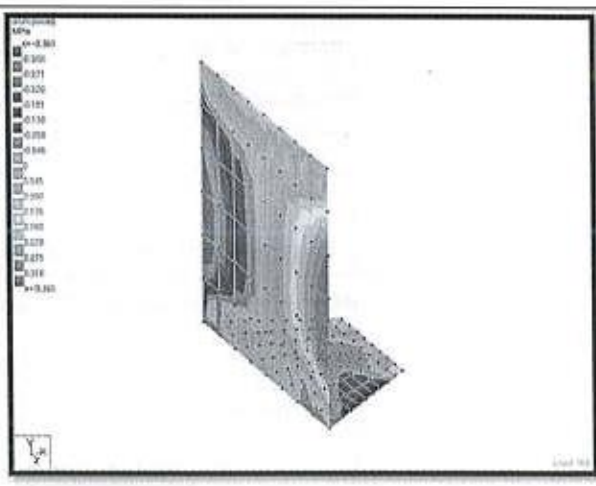

Figure- 25: Shear Force in Breast Wall (SQX)

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DESIGN OF BREAST WALL FOR BENDING MOMENT IN Y DIRECTION (VERTICAL):

Input from STAAD FILE

Maximum Bending moment in Y direction	425	KN-m/m
Maximum Bending moment in Y direction for maximum are	425	KN-m/m

Maximum bending moment in Y direction is occurred at point on the corner, for resist this moment extra oblique bar is provided at the corner, however breast wall is designed for average maximum bending moment in pier i.e 550 KN-m/m

Design

Fy	500	N/mm²
Fck	25	N/mm²
Clear Cover	50	mm
Diameter of bar	20	mm
Width	1000	mm
Thickness of Breast wall	750	mm
Effective depth	690	mm
Design Moment Mu	425	KN-m/m
Mu/b*d^2	0.89	
Pt Required $Pt=\left(\frac{Fck}{2\times Fy}\right)\left(1-\sqrt{1-4.598\times R/Fck}\right)$	0.21	
Ast required	1480	OK
Spacing required using 20 dia bars	212	mm
Spacing Provided	150	mm
Minimum reinforcement @ 0.12% of cross section area	828	mm2
Ast provided	2094	mm2

Provided vertical reinforcement φ20 @ 150 mm c/c spacing

DESIGN OF BREAST WALL FOR BENDING MOMENT IN X DIRECTION (ACROSS THE FLOW):

Input from STAAD FILE

Maximum Bending moment in X direction	454	KN-m/m

Design

Fy	500	N/mm²
Fck	25	N/mm²
Clear Cover	50	mm
Diameter of bar	25	mm
Width	1000	mm
Thickness of Breast wall	750	mm
Effective depth	687.5	mm
Design Moment Mu	454	KN-m/m
Mu/b*d^2	0.96	
Pt Required $Pt=\left(\frac{Fck}{2\times Fy}\right)\left(1-\sqrt{1-4.598\times R/Fck}\right)$	0.23	
Ast required	1591	OK
Spacing required using 25 dia bars	308	mm
Spacing Provided	150	mm
Minimum reinforcement @ 0.12% of cross section area	825	mm2
Ast Provided	3272	mm2

Provided across the reinforcement φ25 @ 150 mm c/c spacing

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F. Bending Moment for Top Slab in X & Y Direction:

			Shear (Local)		Bending Moment (Local)		
	Plate	L/C	SQX MPa	SQY MPa	MX kN-	MY kN-	MXY kN-
Max Qx	1661	101 OPERATION CONDITION-1	0.226	-0.037	-139.038	-25.753	19.697
Min Qx	1719	105 OPERATION CONDITION + EQ(+X)	-0.118	-0.041	-5.601	2.771	3.209
Max Qy	1630	103 CONSTRUCTION CONDITION-1 (EMPTY INTAKE)	0.004	0.368	-17.266	-83.516	-19.165
Min Qy	1720	103 CONSTRUCTION CONDITION-1 (EMPTY INTAKE)	-0.029	-0.349	-13.364	-81.509	13.071
Max Sx	1625	105 OPERATION CONDITION + EQ(+X)	-0.039	-0.053	-1.44	28.07	-0.465
Min Sx	1631	109 CONSTRUCTION CONDITION + EQ (+Z)	0.055	-0.068	-18.391	-9.988	8.147
Max Sy	1680	105 OPERATION CONDITION + EQ(+X)	-0.006	-0.017	1.606	19.377	2.301
Min Sy	1720	107 OPERATION CONDITION + EQ (-Z)	-0.039	-0.258	-9.537	-56.931	10.026
Max Sxy	1639	105 OPERATION CONDITION + EQ(+X)	-0.009	0.039	-6.451	14.773	-13.042
Min Sxy	1709	105 OPERATION CONDITION + EQ(+X)	-0.016	-0.041	-3.871	10.806	15.297
Max Mx	1666	103 CONSTRUCTION CONDITION-1 (EMPTY INTAKE)	0.031	-0.011	25	31.753	-3.987
Min Mx	1671	105 OPERATION CONDITION + EQ(+X)	0.195	0.004	-149	-25.942	-4.27
Max My	1670	103 CONSTRUCTION CONDITION-1 (EMPTY INTAKE)	-0.01	0.03	2.108	48	2.998
Min My	1630	103 CONSTRUCTION CONDITION-1 (EMPTY INTAKE)	0.004	0.368	-17.266	-84	-19.165
Max Mxy	1641	105 OPERATION CONDITION + EQ(+X)	0.131	-0.083	-72.038	-36.99	40.595
Min Mxy	1691	105 OPERATION CONDITION + EQ(+X)	0.143	0.065	-82.451	-28.249	-35.669

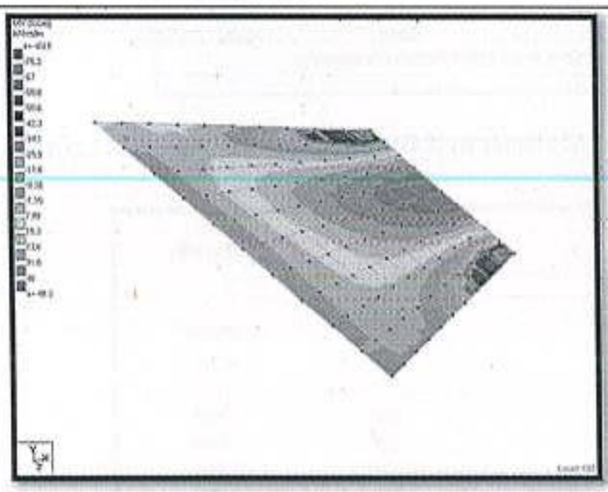

Figure- 26: Maximum Bending Moment (My) Top Slab

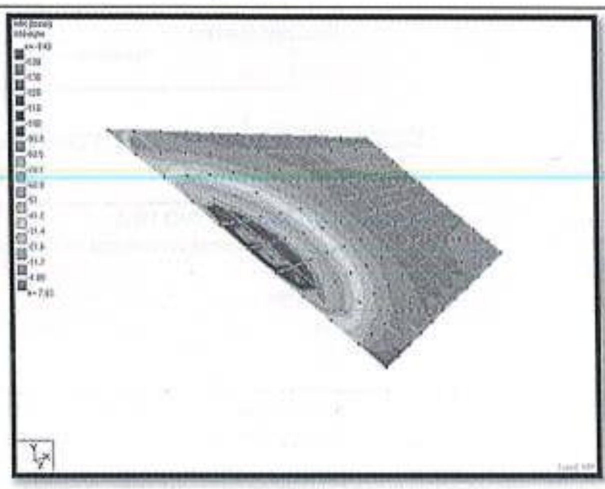

Figure- 27: Maximum Bending Moment (Mx) Top Slab

DESIGN OF TOP SLAB FOR BENDING MOMENT IN Y DIRECTION (ACROSS THE FLOW):

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Input from STAAD FILE		
Maximum Bending moment in Y direction	84	KN-m/m
Design		
Fy	500	N/mm^2
Fck	25	N/mm^2
Clear Cover	25	mm
Diameter of bar	16	mm
Width	1000	mm
Thickness of slab	500	mm
Effective depth	467	mm
Design Moment Mu	84	KN-m/m
Mu/b*d^2	0.38	
Pt required $Pt=\left(\frac{Fck}{2\times Fy}\right)(1-\sqrt{1-4.598\times R/Fck})$	0.09	
Ast required	419	NOT OK
Spacing required using 16 dia bars	480	mm
Minimum reinforcement @ 0.12% of cross section area	560.4	mm2
Dia	16	mm
Spacing	200	mm
No of bar in 1 m	5	nos
Actual Ast provided	1004.8	mm2
	OK	
Provided across the flow reinforcement φ16 @ 200 mm c/c spacing		

DESIGN OF TOP SLAB FOR BENDING MOMENT IN X DIRECTION (ALONG THE FLOW):

Input from STAAD FILE		
Maximum Bending moment in X direction	149	KN-m/m
Design		
Fy	500	N/mm^2
Fck	25	N/mm^2
Clear Cover	25	mm
Diameter of bar	16	mm
Width	1000	mm
Thickness of slab	500	mm
Effective depth	467	mm
Design Moment Mu	149	KN-m/m
Mu/b*d^2	0.68	
Pt required $Pt=\left(\frac{Fck}{2\times Fy}\right)(1-\sqrt{1-4.598\times R/Fck})$	0.16	
Ast required	760	OK
Spacing required using 16 dia bars	265	mm
Minimum reinforcement @ 0.12% of cross section area	560.4	mm2
Dia	16	mm
Spacing	200	mm
No of bar in 1 m	5	nos.
Actual Ast provided	1004.8	mm2
	OK	
Provided along the flow reinforcement φ16 @ 200 mm c/c spacing		

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G. Bending Moment for Backwall in X & Y Direction:

	Plate	L/C	Shear (Local) SQX MPa	SQY MPa	Bending Moment (Local) MX kN-	MY kN-	MXY kN-
Max Qx	2142	107 OPERATION CONDITION + EQ (-Z)	0.396	-0.106	-206.002	-45.146	-14.208
Min Qx	2133	106 OPERATION CONDITION + EQ (+Z)	-0.388	-0.09	-210.36	-45.605	13.288
Max Qy	2143	106 OPERATION CONDITION + EQ (+Z)	0.036	0.553	-138.332	-523.744	-9.583
Min Qy	2152	107 OPERATION CONDITION + EQ (-Z)	0.064	-0.529	-113.62	-469.668	9.604
Max Sx	2133	106 OPERATION CONDITION + EQ (+Z)	-0.388	-0.09	-210.36	-45.605	13.288
Min Sx	2133	110 CONSTRUCTION CONDITION + EQ (-Z)	0.058	-0.021	62.322	4.083	-8.554
Max Sy	2152	106 OPERATION CONDITION + EQ (+Z)	-0.006	0.375	15.679	570.369	-30.669
Min Sy	2133	107 OPERATION CONDITION + EQ (-Z)	-0.118	-0.055	-34.719	-24.217	-3.677
Max Sxy	2133	107 OPERATION CONDITION + EQ (-Z)	-0.118	-0.055	-34.719	-24.217	-3.677
Min Sxy	2141	106 OPERATION CONDITION + EQ (+Z)	0.049	0.029	4.049	-15.501	-13.869
Max Mx	2117	105 OPERATION CONDITION + EQ(+X)	-0.032	-0.02	105	34.488	-3.217
Min Mx	2123	106 OPERATION CONDITION + EQ (+Z)	-0.349	-0.039	-228	-49.353	13.326
Max My	2152	106 OPERATION CONDITION + EQ (+Z)	-0.006	0.375	15.679	570	-30.669
Min My	2143	106 OPERATION CONDITION + EQ (+Z)	0.036	0.553	-138.332	-524	-9.583
Max Mxy	2144	107 OPERATION CONDITION + EQ (-Z)	-0.115	-0.236	-0.359	326.626	61.139
Min Mxy	2151	106 OPERATION CONDITION + EQ (+Z)	-0.124	0.271	13.846	365.537	-63.971

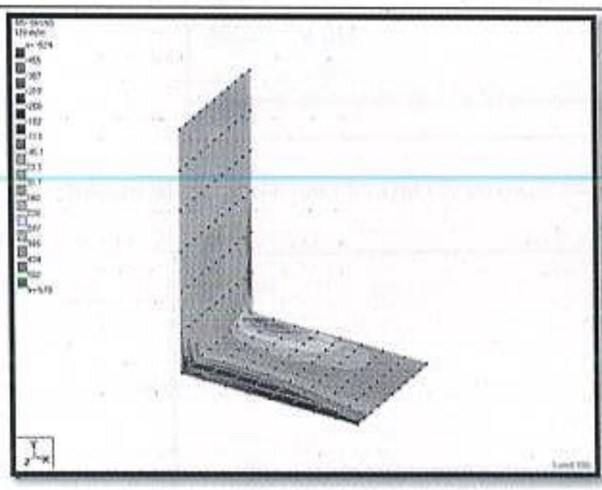

Figure- 28: Maximum Bending Moment (My) Backwall

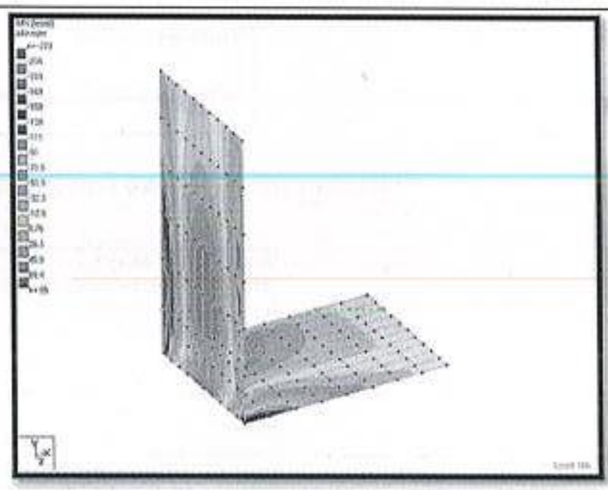

Figure- 29: Maximum Bending Moment (Mx) Backwall

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DESIGN OF BACKWALL FOR BENDING MOMENT IN Y DIRECTION (ACROSS THE FLOW):

Input from STAAD FILE		
Maximum Bending moment in Y direction	570	KN-m/m
Avearge BM	380	KN-m/m
Design		
Fy	500	N/mm^2
Fck	25	N/mm^2
Clear Cover	25	mm
Diameter of bar	20	mm
Width	1000	mm
Thickness of slab	500	mm
Effective depth	465	mm
Design Moment Mu	380	KN-m/m
Mu/b*d^2	1.76	
Pt required $Pt=\left(\frac{Fck}{2\times Fy}\right)\left(1-\sqrt{1-4.598\times R/Fck}\right)$	0.44	
Ast required	2062	OK
Spacing required using 16 dia bars	152	mm
Minimum reinforcement @ 0.12% of cross section area	558	mm2
Ast provided	20	mm
Spacing required using φ20	150	mm
No of bar in 1 m	7	nos
Actual Ast provided	2093	mm2
	OK	

Provided across the flow reinforcement φ20 @ 150 mm c/c spacing

DESIGN OF BACKWALL FOR BENDING MOMENT IN X DIRECTION (ALONG THE FLOW):

Input from STAAD FILE		
Maximum Bending moment in X direction	228	KN-m/m
Design		
Fy	500	N/mm^2
Fck	25	N/mm^2
Clear Cover	25	mm
Diameter of bar	20	mm
Width	1000	mm
Thickness of slab	500	mm
Effective depth	465	mm
Design Moment Mu	228	KN-m/m
Mu/b*d^2	1.05	
Pt required $Pt=\left(\frac{Fck}{2\times Fy}\right)\left(1-\sqrt{1-4.598\times R/Fck}\right)$	0.26	
Ast required	1187	OK
Spacing required using 20 dia bars	264	mm
Minimum reinforcement @ 0.12% of cross section area	558	mm2
Dia	20	mm
Spacing	150	mm
No of bar in 1 m	7	nos.
Actual Ast provided	2093	mm2
	OK	

Provided along the flow reinforcement φ20 @ 150 mm c/c spacing

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H. DESIGN FOR SHEAR FORCE (SQX & SQY) FOR PIER, RAFT, TOP SLAB AND BREAST WALL

	Concrete		25.00	Mpa						
	Steel		415.00	Mpa						
Part	Type	Shear [MPa]	Reinforcement (Pt %)	Shear Capacity [MPa]	Design Shear [Mpa]	Dia [mm]	Thickness [mm]	L, Longi. Spacing (mm)	B, Lateral Spacing (mm)	Check
Raft	SQx	0.61	0.11	0.252	0.36	12.0	1,200.0	250.0	250.0	OK
Raft	SQy	0.39	0.14	0.287	0.11	12.0	1,200.0	250.0	250.0	OK
Top Slab	SQx	0.23	0.16	0.303	-	-	-	-	-	Not Required
Top Slab	SQy	0.37	0.09	0.231	0.14	12.0	500.0	-	-	Not Required
Pier Zone-1	SQx	0.30	0.20	0.329	-	-	-	-	-	Not Required
Pier Zone-1	SQy	0.32	0.21	0.339	-	-	-	-	-	Not Required
Pier Zone-2A	SQx	0.45	0.13	0.272	0.18	12.0	2,000.0	250.0	250.0	OK
Pier Zone-2A	SQy	0.47	0.20	0.334	0.14	12.0	2,000.0	250.0	250.0	OK
Pier Zone-2B	SQx	0.49	0.11	0.249	0.24	12.0	2,000.0	250.0	250.0	OK
Pier Zone-2B	SQy	0.35	0.07	0.199	0.15	12.0	2,000.0	250.0	250.0	OK
Breast wall	SQx	0.25	0.16	0.303	-	-	-	-	-	Not Required
Breast wall	SQy	0.15	0.21	0.342	-	-	-	-	-	Not Required