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DESIGN BASIS REPORT FOR

PROPOSED RESIDENTIAL TOWER FOR

M/s. RAJPARIS AT MUTTUKADU, CHENNAI.

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Door No.50, Kirupasankari 2nd Street,
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1.0 OBJECTIVE

The Design basis covers the basic requirement that will be followed for the design of Proposed Residential Tower for M/s. RAJPARIS at MUTTUKADU, CHENNAI.

The building is designed for Basement + Stilt + 14 Floors. The structure is supported on columns in turn resting on Raft and Pile foundation. The structural system comprises columns, beams and slab structure.

3-D Analysis of all the building structures will be carried out using STAAD PRO software package and analysis results shall be used for designing the various elements.

2.0 DESIGN LOADS

2.1 Dead Loads

The Dead loads that are envisaged to act permanently are considered as per IS: **875 (Part 1) – 1987 – Dead loads.** Indian Standard **"Code of Practice for Design Loads (other than earthquake) for Buildings and Structures"**

1. Slab is assumed to be 125 mm thick – 3.125 KN/m²
2. Slab is assumed to be 140 mm thick – 3.5 KN/m²
3. Slab is assumed to be 165 mm thick – 4.125 KN/m²
4. Slab is assumed to be 175 mm thick – 4.375 KN/m²
5. Slab is assumed to be 250 mm thick – 6.25 KN/m²
6. Floor finish – 1.25 KN/m²
7. Brick step load – 4 kN/m²
8. AAC Block walls of 200 & 100 mm thick are proposed to be used (location of walls as per Architectural drawings) Unit weight is taken as 8.5 KN/m³

2.2 Imposed Loads

The imposed loads that are envisaged to act permanently are considered as per **IS : 875 (Part 2) – 1987** . Indian Standard **"Code of Practice for Design Loads (other than earthquake) for Buildings and Structures"**.

1. All rooms = 2 KN/m²
2. Terrace = 1.5 kN/m²

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2.3 Wind Load

The wind pressure shall be calculated based on the provision laid in **IS : 875 (Part 3) – 2015.** The basic wind speed, terrain category, height factor and structures class etc. required for the wind pressure calculation can be furnished from this codal provisions.

Location	:	Chennai		
Wind Loads	:	As per IS 875		
As per IS 875 Part 3	:	Basic Wind speed	V_b	= 50 m/Sec.
		(Table 1)	K1	= 1.00
		(Table 2)	K2	= Terrain category -1

Class – B

0 to 10m – K2 = 1.05

10 to 15m – K2 = 1.09

15 to 20m – K2 = 1.12

20 to 30m - K2 = 1.15

30 to 50m - K2 = 1.20

(Table 3) K3 = 1.0

K4 = 1.0

Design Wind Speed (V_z) : V_b x K1 x K2x K3xK4

: 50 x 1.00 x 1.05 x 1.00 x 1.00 = 52.5 m/Sec.

Wind Pr : 0.6 x Vz^2

Pz : 0.6 x 52.5^2 = 1.65 kN /m^2

Kd = 0.9

Ka = 0.9

Kc = 0.9

Design Wind Pr. Pd : Pz x Kd x Ka x Kc

1.65x0.9x0.9x0.9 = 1.21 kN /m²

Design Wind Speed (V_z) : V_b x K1 x K2x K3xK4

: 50 x 1.00 x 1.09 x 1.00 x 1.00 = 54.5 m/Sec.

Wind Pr : 0.6 x Vz^2

Pz : 0.6 x 54.5^2 = 1.78 kN /m²

Kd = 0.9

Ka = 0.9

Kc = 0.9

Design Wind Pr. Pd : Pz x Kd x Ka x Kc

54.5x0.9x0.9x0.9 = 1.3 kN /m²

Design Wind Speed (V_z) : V_b x K1 x K2x K3xK4

: 50 x 1.00 x 1.12 x 1.00 x 1.0 = 56 m/Sec.

Wind Pr : 0.6 x Vz^2

Pz : 0.6 x 56^2 = 1.88 kN /m²

Kd = 0.9

Ka = 0.9

Kc = 0.9

Design Wind Pr. Pd : Pz x Kd x Ka x Kc

1.88x0.9x0.9x0.9 = 1.37 kN /m²

Design Wind Speed (V_z) : V_b x K1 x K2x K3xK4

: 50 x 1.00 x 1.15 x 1.00 x 1.00 = 57.5 m/Sec.

Wind Pr : 0.6 x Vz^2

Pz : 0.6 x 57.5^2 = 1.98 kN /m²

	Kd	=	0.9
	Ka	=	0.9
	Kc	=	0.9
Design Wind Pr. Pd	:	Pz x Kd x Ka x Kc	
	1.98x0.9x0.9x0.9	=	1.45 kN /m²

Design Wind Speed (V_z)	:	V_b x K1 x K2x K3xK4	
	:	50 x 1.00 x 1.20 x 1.00 x 1.00	= 60 m/Sec.
Wind Pr	:	0.6 x Vz²	
Pz	:	0.6 x 60²	= 2.16 kN /m²

	Kd	=	0.9
	Ka	=	0.9
	Kc	=	0.9
Design Wind Pr. Pd	:	Pz x Kd x Ka x Kc	
	2.16x0.9x0.9x0.9	=	1.57 kN /m²

2.4 Earthquake Load

The loading due to earthquake is assessed based on the provisions of **IS : 1893-2016**. Structure with ductile detailing is considered as per **IS : 13920-2016.**

Chennai falls under Seismic Zone III

Zone factor = 0.16

Response reduction factor = 5

Importance factor = 1.5

Structure with ductile detailing is considered as per IS: 13920-2016

As per **IS: 1893-2002**, percentage of imposed loads to be considered for seismic weight calculation is mentioned below :

Imposed Uniformity Distributed Floor Load Loads (kN/ m^2)	Percentage of Imposed
Up to & including 3.0	25
Above 3.0	50

2.5 Load Combinations

The various loads shall be combined in accordance with the stipulations in IS: 875 (Part 5) – 1987. Whichever combinations produced the most unfavorable effect in the building, foundations or structural member concerned shall be adopted. The various load combinations considered in the analysis are as follows:

LOAD COMB 100 1.5(DL+LL)

3 1.5 4 1.5

LOAD COMB 101 1.2(DL+LL+SX)

3 1.2 4 1.2 1 1.2

LOAD COMB 102 1.2(DL+LL+S-X)

3 1.2 4 1.2 1 -1.2

LOAD COMB 103 1.2(DL+LL+SZ)

3 1.2 4 1.2 2 1.2

LOAD COMB 104 1.2(DL+LL+S-Z)

3 1.2 4 1.2 2 -1.2

LOAD COMB 105 1.5(DL+SX)

3 1.5 1 1.5

LOAD COMB 106 1.5(DL-SX)

3 1.5 1 -1.5

LOAD COMB 107 1.5(DL+SZ)

3 1.5 2 1.5

LOAD COMB 108 1.5(DL-SZ)

3 1.5 2 -1.5

LOAD COMB 109 0.9DL+1.5SX

3 0.9 1 1.5

LOAD COMB 110 0.9DL-1.5SX

3 0.9 1 -1.5

LOAD COMB 111 0.9DL+1.5SZ

3 0.9 2 1.5

LOAD COMB 112 0.9DL-1.5SZ

3 0.9 2 -1.5

LOAD COMB 113 1.2(DL+LL+WL +X)

3 1.2 4 1.2 5 1.2

LOAD COMB 114 1.2(DL+LL+WL -X)

3 1.2 4 1.2 6 1.5

LOAD COMB 115 1.2(DL+LL+WL +Z)

3 1.2 4 1.2 7 1.5

LOAD COMB 116 1.2(DL+LL+WL -Z)

3 1.2 4 1.2 8 1.5

LOAD COMB 117 1.5(DL+WL +X)

3 1.5 5 1.5

LOAD COMB 118 1.5(DL+WL -X)

3 1.5 6 1.5

LOAD COMB 119 1.5(DL+WL +Z)

3 1.5 7 1.5

LOAD COMB 120 1.5(DL-WL -Z)

3 1.5 8 1.5

****

LOAD COMB 200 1.0(DL+LL)

3 1.0 4 1.0

LOAD COMB 201 1.0DL+0.8LL+0.8SX

3 1.0 4 0.8 1 0.8

LOAD COMB 202 1.0DL+0.8LL-0.8SX

3 1.0 4 0.8 1 -0.8

LOAD COMB 203 1.0DL+0.8LL+0.8SZ

3 1.0 4 0.8 2 0.8

LOAD COMB 204 1.0DL+0.8LL-0.8SZ

3 1.0 4 0.8 2 -0.8

LOAD COMB 205 1(DL+SX)

3 1.0 1 1.0

LOAD COMB 206 1(DL-SX)

3 1.0 1 -1.0

LOAD COMB 207 1(DL+SZ)

3 1.0 2 1.0

LOAD COMB 208 1(DL-SZ)

3 1.0 2 -1.0

LOAD COMB 209 1.0DL+0.8LL+0.8WL +X

3 1.0 4 0.8 5 0.8

LOAD COMB 210 1.0DL+0.8LL-0.8WL -X

3 1.0 4 0.8 6 0.8

LOAD COMB 211 1.0DL+0.8LL+0.8WL +Z

3 1.0 4 0.8 7 0.8

LOAD COMB 212 1.0DL+0.8LL-0.8WL -Z

3 1.0 4 0.8 8 0.8

LOAD COMB 213 1(DL+WL +X)

3 1.0 5 1.0

LOAD COMB 214 1(DL+WL -X)

3 1.0 6 1.0

LOAD COMB 215 1(DL+WL +Z)

3 1.0 7 1.0

LOAD COMB 216 1(DL+WL -Z)

3 1.0 8 1.0

The above load combinations will be considered and effect of worst combinations will be taken for design of various building elements.

3.0 ANALYSIS METHODS

The analysis of the structures is carried out using the STAAD PRO. The entire super-structure is modeled using frame elements as appropriate. Beams and columns are modeled as frame elements. Appropriate loads and its combination as per relevant IS code clauses are chosen for design.

4.0 DESIGN ASSUMPTION

4.1.1 The Base of the Foundation is assumed to be FIXED.

4.1.2 All Concrete elements shall be designed in accordance with IS 456: 2000 Limit State Design.

5.0 FOUNDATION DESIGN

1. Type of Foundation : Pile Foundation. (as per soil report)
2. Capacity 750dia : 1350kN
3. Capacity 600dia : 800kN
4. Capacity 500dia : 650kN
5. Capacity 450dia : 500kN
6. Capacity 400dia : 400kN

6.0 MATERIALS

The self-weight of the various elements is computed based on the unit weight of materials as given below:

S.no	Materials	Unit weight KN/m^3
1	Reinforced Cement Concrete	25
2	Solid Block / Brick	20
3	Steel	78.50

Structural concrete shall be of grade M30 for Pile.

Structural concrete shall be of grade M25 for all other structural Elements.

Steel reinforcement shall be of Grade Fe500 confirming to IS: 1786-2008.

7.0 NOMINAL COVER TO REINFORCEMENT

From Durability requirement, exposure condition is assumed as mild for structural elements above ground level and moderate for structural elements below ground level.

The Clear cover to reinforcement to meet durability requirement shall be as follows:

Column	–	40mm
Pile	–	50mm
Pile cap top	–	50mm
Pile cap bottom	–	100mm
Beam	–	25mm
Slab	–	20mm

8.0 LIST OF CODES & REFERENCES:

- IS: 456 2000: Code of practice for Plain and Reinforced Concrete.
- IS: 875 Part 1: Code of Practice for Design of Dead Loads for buildings and structures.
- IS: 875 Part 2: Code of Practice for Design of imposed Loads for buildings.
- IS: 875 Part 3: Code of Practice for Design Loads (Wind loads) for buildings & structures (other than Earthquake).
- IS: 1893 (Part 1)-2016: Criteria for Earthquake resistant Design of structures.
- IS: 13920-2016: Ductile detailing of reinforced concrete structures subjected to seismic forces.
- IS 1786-1985: code of practice for steel.
- SP 16: Design aids for reinforced concrete to IS: 456- 1978.
- SP 34: Hand book on concrete reinforcement detailing.

Rajparis_Muttukadu					
DESIGN OF COLUMN - C1					
Length of column			D	1200	mm
Breadth of column			b	300	mm
Unsupported length (longer direction)			L_x	3200	mm
Unsupported length (shorter direction)			L_y	3200	mm
Eff. Length (longer direction)			L_{ex}	2560	mm
Eff. Length (shorter direction)			L_{ey}	2560	mm
Factored Load			P_u	4172	kN
Factored moment (longer direction)			Mux	318	kNm
Factored moment (shorter direction)			Muz	30	kNm
Charac. comp strength of concrete			f_{ck}	30	N/mm^2
Yield stress of steel			f_y	500	N/mm^2
Dia of bar used			Φ	25	mm
Clear cover				45	mm
Effective cover			d'	57.5	mm
Effective depth			d	1142.5	mm
Check for slenderness					
Longer Direction					
Slenderness ratio			L_{ex}/D	2.13	**short**
Additional moment due to slenderness			M_{ax}	0	kN-m
Shorter Direction					
Slenderness ratio			L_{ey}/b	8.53	**short**
Additional moment due to slenderness			M_{ay}	0	kN-m
Calculation of P_{uz}					
No of bars used				24	
Area of reinforcement			Ast	11775	
Provided steel				24 nos Y25	
Percentage of reinforcement			p_t	3.271	%
Area of concrete			Ag	360000	mm^2
Ultimate axial load (without moments)			P_{uz}	9117	kN
Calculation of P_b:					
Longer Direction					
			d'/D	0.048	
Coefficient k_1(from TABLE 60 of SP:1980)			k_{1x}	0.219	From Sp16
Coefficient k_2(from TABLE 60 of SP:1980)			k_{2x}	0.545	
Axial load corresponding to balanced failure			P_{bx}	3007	kN
Reduction factor for additional moments			k_x	1.000	
Moment due to slenderness			M_{ax}	**0.00**	kN-m
Shorter Direction					
			d'/b	0.192	
Coefficient k_1(from TABLE 60 of SP:1980)			k_{1y}	0.185	
Coefficient k_2(from TABLE 60 of SP:1980)			k_{2y}	0.205	
Axial load corresponding to balanced failure			P_{by}	2239	kN
Reduction factor for additional moments			k_y	0.719	

Moment due to slenderness		M_{ax}	**0.00**	kN-m	
Check for moments due to min. eccentricity					
Longer Direction					
Minimum eccentricity		e_x	46.40	mm	
Moment due to min. eccentricity		M_x	193.58	kN-m	
Moment in longer direction		M_{ux}	318.00	kNm	
Shorter direction					
Minimum eccentricity		e_y	16.40	mm	
Moment due to min. eccentricity		M_y	68.42	kN-m	
Moment in shorter direction		M_{uy}	68.4208	kNm	
Final design moments					
Moment in longer direction		M_{ux}	**318.00**	kNm	
Moment in shorter direction		M_{uy}	**68.42**	kNm	
Check for interaction					
		d'/D	0.048		
		d'/b	0.192		
p/f_{ck}		p/f_{ck}	0.109		
$P_u/f_{ck}bD$ value		$P_u/f_{ck}bD$	0.386		
$M_{ux1}/f_{ck}bD^2$ value (from SP 16:1980 charts)		$M_{ux1}/f_{ck}bD^2$	0.110		
Uniaxial moment capacity (longer direction)		M_{ux1}	**1425.6**	kNm	
$M_{uy1}/f_{ck}bD^2$ value (from SP 16:1980 charts)		$M_{uy1}/f_{ck}bD^2$	0.110		
Uniaxial moment capacity (shorter direction)		M_{uy1}	**356.4**	kNm	
P_u/P_{uz} ratio		P_u/P_{uz}	**0.458**		
Value of exponent			1.430		
Interaction ratio			**0.211**		
Percentage reinforcement satisfactory or not			**safe**		

DESIGN OF TYPICAL FLOOR SLAB

CL. NO.	DESIGN CALCULATIONS

1.0 Design of two way Slab — **S1**

1.0.1 Basic Data

Grade of concrete (fck)		=	M 25	
Grade of steel (fy)		=	Fe 500	
Thickness of slab (D)		=	125	mm
Clear cover (cc)		=	20	mm
Effective thickness of slab (deff)	= 125 - 20 - 10 /2	=	100	mm
Unit weight of concrete		=	25	KN/m^3
Effective Shorter Span (Lx)		=	4.1	m
Effective Longer Span (Ly)		=	5.1	m
Aspect ratio Ly/Lx		=	1.24	**Two way slab**
Edge Condition :	**Interior panels**			

1.0.2 Load Data

Dead Load	= 0.13 x 25	=	3.125	KN/m^2
Floor finish		=	1.25	KN/m^2
Imposed Load		=	2.00	KN/m^2
Total load		=	6.38	KN/m^2
Design load (Wu) = 1.5 x 6.375		=	**9.56**	KN/m^2

1.0.3 Moment and Shear Calculations

Short Span Direction

ax for positive moment			1.2	→	0.032	
			1.3	→	0.036	
		for 1.24	αx	=	0.034	
Positive moment Mx :	0.034 x	9.56 x	4.1 ^2	=	**5.43**	KN-m
ax for negative moment			1.2	→	0.043	
			1.3	→	0.047	
		for 1.24	αx	=	0.045	
Negative moment Mx :	0.045 x	9.56 x	4.1 ^2	=	**7.19**	KN-m

Long Span Direction

For positive moment			αy	=	0.024	
Positive moment My :	0.024 x	9.56 x	4.1 ^2	=	**3.86**	KN-m

For negative moment αy = 0.032

Negative moment My : 0.032 x 9.56 x 4.1 ^2 = **5.14** KN-m

Shear force V_u = 9.56 x 4.1 x 0.5 = **19.60** KN

1.0.4 **Check Effective Depth for Bending**

Depth required = $(M_x / (0.133x\ fck \times b))^{1/2}$ = 46.52 mm

Depth provided = 125 - 20 - 10 /2 = 100 mm

Hence O.K

1.0.5 **Reinforcement calculations**

Short Span Direction @ Support

$$Mu/bd^2 = \frac{7.19 \times 10 \wedge 6}{1000 \times 100 \times 100} = \mathbf{0.719}$$

Pt = **0.17** %

Area of steel Ast = 0.00171 x 1000 x 100 = 171.342 mm^2

Minimum area of steel: 0.0012 x 1000 x 125 = 150.00 mm^2

Spacing of Y **10** bars 78.54 x 1000 / 171.34 = 458.149 mm

Ast provided = 628.32 mm^2

Check for Spacing

Minimum of

1) 3d = 300 mm

2) = 300 mm

3) = 458.15 mm

Therefore provide Y **10** @ **125** mm C/C

Short Span Direction @ Mid Span

$$Mu/bd^2 = \frac{5.43 \times 10 \wedge 6}{1000 \times 100 \times 100} = 0.543$$

Pt = 0.128 %

Area of steel Ast = 0.00128 x 1000 x 100 = 128.082 mm^2

Minimum area of steel: 0.0012 x 1000 x 125 = 150.00 mm^2

Spacing of Y **10** bars 78.54 x 1000 / 150.00 = 523.333 mm

Ast provided = 628.32 mm^2

Check for Spacing

Minimum of

1) 3d = 300 mm

2) = 300 mm

3) = 523.33 mm

Therefore provide Y **10** @ **125** mm C/C 100

Provided % of Tension Reinforcement (pt) = **0.628** %

Long Span Direction @ Support

$Mu/bd^2 = \frac{5.14 \times 10 \wedge 6}{1000 \times 100 \times 100} = 0.514$

Pt = 0.121 %

Area of steel Ast	=	0.00121	x	1000 x	100	=	121.249	mm^2
Minimum area of steel:		0.0012	x	1000 x	125	=	150.00	mm^2
Spacing of Y **10** bars		78.54	x	1000 /	150.00	=	523.333	mm
Ast provided						=	523.6	mm^2

Check for Spacing

Minimum of	1) 3d	=	300	mm
	2)	=	300	mm
	3)	=	523.33	mm

Therefore provide Y **10** @ **150** mm C/C

Long Span Direction @ mid Span

$Mu/bd^2 = \frac{3.86 \times 10 \wedge 6}{1000 \times 100 \times 100} = 0.386$

Pt = 0.090 %

Area of steel Ast	=	0.00090	x	1000 x	100	=	90.3647	mm^2
Minimum area of steel:		0.0012	x	1000 x	125	=	150.00	mm^2
Spacing of Y **10** bars		78.54	x	1000 /	150.00	=	523.333	mm
Ast provided						=	523.6	mm^2

Check for Spacing

Minimum of	1) 3d	=	300	mm
	2)	=	300	mm
	3)	=	523.33	mm

Therefore provide Y **10** @ **150** mm C/C

1.0.6 **Check for Deflection:**

Basic Span/depth Ratio	=	20	
% of Tension Reinforcement provided (pt)	=	0.628	%
Service stress of steel (fs) : 0.58fy(Ast_r/Ast_p)	=	59.12	N/mm^2
Modification Factor for Tension Reinforcement (F1)	=	2.00	
Allowable Span/Eff Depth Ratio	=	42	
Actual Span/Eff Depth Ratio = 4.1 x 1000 / 100	=	41.00	

Hence O.K

Design of beam at Grid 22 **RAJPARIS_MUTTUKADU**

Grade of concrete, f_{ck}			=	25 N/mm^2
Grade of steel,f_y			=	500 N/mm^2
Dia of stirrups			=	8 mm
Clear cover			=	25 mm
Width of web, bw			=	200 mm
Depth of section			=	825 mm
clear span			=	4300 mm

Design for Maximum Sagging Bending Moment

Beam element Number in staad model				
Governing load combination				
ultimate bending Moment at section , M_u			=	112 kNm
ultimate torsional moment,T_u			=	2 kNm
design bending moment			=	118 kNm
Dia of tension bar-1st layer			=	16 mm
Number of tension bars-1st layer			=	2 Nos
Dia of tension bar-2nd layer			=	25 mm
Number of tension bars-2nd layer			=	0 Nos
Ast provided			=	402 mm^2
Effective depth, d			=	792 mm
Dia of bars in compresson-1st layer			=	3 mm
No. of compression bars-1st layer			=	16 nos.
Dia of bars in compresson-2nd layer			=	0 mm
No. of compression bars-2nd layer			=	0 nos.
Area of compression steel			=	113 mm^2
Effective cover to compression reinforcement, d'			=	26.5 mm
Strain in compression steel			=	0.00223
Stress in compression steel		f_{sc}	=	389.1 N/mm^2
Strain in tension steel			=	0.03548
Stress in tension steel		f_{st}	=	434.8 N/mm^2
depth of the nautral axis			=	72.98 mm
				Under reinforecd

Ultimate moment of resistance,

M_{ur} =	$C_{uc}(d-0.416X_u)+C_{us}(d-d')$			
C_{uc} =	$0.362f_{ck}bX_u$	=0.362×25×200×73	=	132097 N
C_{us} =	$(f_{sc}-0.447f_{ck})A_{sc}$	=(389-0.447×25)×113	=	42747 N
T_{us}=	$f_{st}A_{st}$	=434.8×402	=	174843 N
M_{ur} =	=(132097×(792-0.416×73)+42747×(792-26.5))/10^6			
			=	133 kNm
			> Mu :	118 kNm
				ok

MINIMUM REINFORCEMENT REQUIREMENT

Minimum Reinforcement as per IS 456:2000= 0.85/fy=Ast/(bd)		=	0.17	%
Maximum steel on any face at any section		=	4.0	%
Reinforcement Provided at mid span		=	402	mm^2
Provide 2 nos of 16 mm dia bars at bottom	ρ_{pro}	=	0.24	%
Provide 16 nos of 3 mm dia bars at top	ρ_{pro}	=	0.07	%

Development Length

Development Length, $L_d = \phi \sigma_s / 4 \tau_{bd}$

Bond Stress,	τ_{bd}	=	1.5	MPa
60 % increase in bond stress for deformed bars				
Hence, Design Bond Stress		=	2.4	MPa
Grade of steel		=	500.0	Mpa
Permisible stress in bar,	$\sigma_s = 0.87 f_y$	=	435	MPa
L_d	= (435*dia)/(4x2.4)	=	45	ϕ

Check for Serviceability:

Crack Width Calculation

Governing load combination

moment at service load		=	75	kNm
Overall depth of beam D		=	825.00	mm
Effective depth d		=	792.00	mm
Width of the beam b		=	200.00	mm
Diameter of the bar d_b		=	16	mm
Youngs modulus of the reinforcement Es		=	200000	N/mm^2
Short term Modulus of elasticity of concrete		=	25000	N/mm^2
Modulur ratio for tension reinforcement m_t		=	11.20	
Modulur ratio for compression reinforcement m_c		=	16.80	

$0.5bx^2$ +	$(m_c-1)A_{sc}(x-d')$	=	$m_tA_{st}(d-x)$	
100 x^2 +	1786.94 (x- 26.50)	=	4503.79	(792.00 -x)
100 x^2 +	6291 - 3614353	=	0	

Depth of NA, x		=	161.25	mm
Cracked moment of inertia Icr		=	2.10E+09	mm^4
The tensile stress in the reinforcement	fst = m*M*(d-x) / Icr	=	250.73	N/mm^2
Strain in concrete at outermost tension face	e_1	=	0.0013	

$$e_1 = \frac{f_{st}\,(D-x)}{Es\,(d-x)}$$

$$e_2 = \frac{b\,(D-x)^2}{3E_sA_{st}(d-x)}$$

e_2		=	0.0006	
Average steel strain at level considered	$e_1 - e_2$	=	0.0007	> 0
Minimum cover to the longitudinal reinforcement C_{min}		=	25.00	mm

Distance from the point considered to surface of the nearest longitudinal bar

acr(at bottom corner)		=	38.67	mm
acr(at a point on the tension face midway b/w two bars)		=	66.69	mm
acr consider		=	66.69	mm
Actual crack width w_{cr} =	$\frac{3\, a_{cr}\, e_m}{[1+2(a_{cr}-C_{min})/(D-x)]}$	=	0.13	mm
Allowable crack width		=	0.30	mm
		Crack width is within allowable limit		

Design for Maximum Hogging Bending Moment

Beam element Number in staad model				
Governing load combination				
Design bending Moment , M_d		=	20	kNm
ultimate torsional moment,T_u		=	2	kNm
design bending moment		=	26	kNm
Dia of tension bar-1st layer		=	16	mm
Number of tension bars-1st layer		=	3	Nos
Dia of tension bar-2nd layer		=	25	mm
Number of tension bars-2nd layer		=	0	Nos
Ast provided		=	603	mm^2
Effective depth, d		=	792	mm
Dia of bars in compresson-1st layer		=	16	mm
No. of compression bars-1st layer		=	2	nos.
Dia of bars in compresson-2nd layer		=	0	mm
No. of compression bars-2nd layer		=	0	nos.
Area of compression steel		=	402	mm^2
Effective cover to compression reinforcement, d'		=	33	mm
Strain in compression steel		=	0.00181	
Stress in compression steel	f_{sc}	=	355.3	N/mm^2
Strain in tension steel		=	0.03900	
Stress in tension steel	f_{st}	=	434.8	N/mm^2
depth of the nautral axis		=	68.44	mm
		Under reinforecd		

Ultimate moment of resistance,

M_{ur} =	$C_{uc}(d-0.416X_u)+C_{us}(d-d')$			
C_{uc} =	$0.362f_{ck}bX_u$	=0.362×25×200×68	= 123878	N
C_{us} =	$(f_{sc}-0.447f_{ck})A_{sc}$	=(355-0.447×25)×402	= 138388	N
T_{us}=	$f_{st}A_{st}$	=434.8×603	= 262265	N
M_{ur} =	=(123878×(792-0.416×68)+138388×(792-33))/10^6			
			= 200	kNm
			> Mu : 26	kNm
			ok	

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MINIMUM REINFORCEMENT REQUIREMENT			
Minimum Reinforcement as per IS 456:2000= 0.85/fy=Ast/(bd)	=	0.17	%
Maximum steel on any face at any section	=	4.0	%
Reinforcement Provided for support	=	603	mm^2
Provide 3 nos of 16 mm dia bars at top	=	0.37	%
Provide 2 nos of 16 mm dia bars at bottom	=	0.24	%
Shear Design			
Beam element Number in staad model			
Governing load combination			
Design Shear force, Vu	=	145	kN
Torsion, Tu	=	0	kNm
Equivalent shear, Ve	=	145	kN
Effective Depth ,d	=	792	mm
Width of section,b	=	200	mm
Nominal Shear stress $\tau_v = v_u / (bxd)$	=	0.92	N/mm^2
Percentage of reinforcement	=	0.38	%
Design Shear stress τ_c	=	0.44	N/mm^2
the maximum shear stress	=	3.2	N/mm^2
	Shear Reinforcement required		
$v_u - \tau_c bd$	=	75.83	kN
No of legs	=	2	
Area of stirrups	=	101	mm^2
Required Spacing of stirrups	=	457	mm
Maximum spacing	=	300	mm
Provided spacing	=	100	mm
Provide 2 legged 8 mm dia bar at 100 mm c/c			
		ok	
Check for Serviceability:			
Crack Width Calculation			
Governing load combination			
moment at service load	=	13	kNm
Overall depth of beam D	=	825.00	mm
Effective depth d	=	792.00	mm
Width of the beam b	=	200.00	mm
Diameter of the bar d_b	=	16	mm
Youngs modulus of the reinforcement Es	=	200000	N/mm^2
Short term Modulus of elasticity of concrete	=	25000	N/mm^2
Modulur ratio for tension reinforcement m_t	=	11.20	

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Modulur ratio for compression reinforcement m_c					=	16.80	
$0.5bx^2$ +	$(m_c-1)A_{sc}(x-d')$	=	$m_tA_{st}(d-x)$				
100 x^2 +	6353.56 (x- 33.00)	=	6755.68	(792.00 -x)			
100 x^2 +	13109 - 5560167	=	0				
Depth of NA, x					=	179.19	mm
Cracked moment of inertia Icr					=	3.06E+09	mm^4
The tensile stress in the reinforcement		fst = m*M*(d-x) / Icr			=	29.94	N/mm^2
Strain in concrete at outermost tension face				e_1	=	0.0002	
e_1 =	$\frac{f_{st}(D-x)}{Es(d-x)}$						
e_2 =	$\frac{b(D-x)^2}{3E_sA_{st}(d-x)}$			e_2	=	0.0004	
Average steel strain at level considered				$e_1 - e_2$	=	-0.0002 < 0	
Minimum cover to the longitudinal reinforcement C_{min}					=	25.00	mm
Distance from the point considered to surface of the nearest longitudinal bar							
acr(at bottom corner)					=	38.67	mm
acr(at a point on the tension face midway b/w two bars)					=	66.69	mm
acr consider					=	66.69	mm
Actual crack width w_{cr} =	$\frac{3\,a_{cr}\,e_m}{[1+2(a_{cr}-C_{min})/(D-x)]}$				=	-0.04	mm
Allowable crack width					=	0.30	mm

Negative sign shows that crack is not for

on behalf of DICP HRB panel, the structural design calculations are generally reviewed and is in order

12/3/25

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