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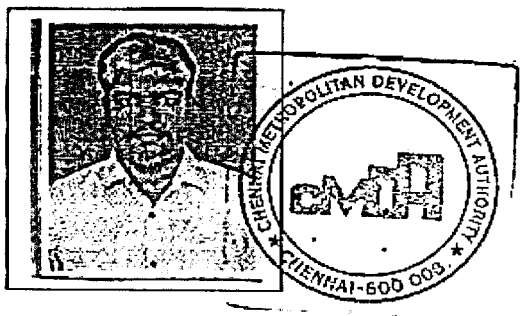
CHENNAI METROPOLITAN DEVELOPMENT AUTHORITY
'ThalamuthuNatarajan Building', No.1, Gandhi Irwin Road,
Egmore, Chennai-600 008.

PROFESSIONAL REGISTRATION CERTIFICATE

This is to certify that the Thiru. N. KARIKALAN has enrolled as Registered Structural Engineer Grade-I (SE) with Chennai Metropolitan Development Authority (CMDA) who has the qualification of M.E.,(Structural Engg.) and experience in the relevant field for **10years**.

He has been given the Registration No. SE/GR-I/2019/03/012 Renewed on **01.03.2024.** He shall be required to comply with the duties and responsibilities prescribed by the Government of Tamil Nadu in Part-V, "Registration of Professionals", Tamil Nadu Combined Development and Building Rules, 2019 approved vide G.O.(Ms) No.18 MAWS (MA1) Dept. dated 04.02.2019.

This Registration is valid upto **29.02.2029.**



(A. MEGANATHAN)
Superintendent

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DATE	REV. NO.	DESCRIPTION	DESIGNED	CHECKED	APPROVED
02.08.2024	0	ISSUED FOR APPROVAL	JSAN	NKK	NKK

REVISIONS

CLIENT :	**PROJECT MANAGEMENT &CONSULTANT:**
M/S BUILD TECH ASSOCIATES	
PROJECT :	**PROPOSED RESIDENTIAL BUILDING AT VGN LAKSHMI NAGAR, KOLAPAKKAM**
SUPPLIER / CONTRACTOR:	
JOB No.	A23015

TOTAL NO. OF PAGES				**TITLE :**	**STRUCTURAL DESIGN DOCUMENT**
	NAME	**SIGN**	**DATE**		**CODE**
DSGN	JSAN	JSAN	02.08.2024	**DESIGN CALCULATIONS (STILT+ 4 FLOORS ONLY)**	IS
CHKD	NKK	NKK	02.08.2024		**REV.**
APPD	NKK	NKK	02.08.2024		0

Doc. No.	A	2	3	0	1	5	-	C	-	R	B	-	0	1	-	D	C	-	0	0	0	1

RELEASED FOR ☐ **PRELIMINARY** ☑ **APPROVAL** ☐ **CONSTRUCTION**

PROJECT:	PROPOSED RESIDENTIAL BUILDING AT VGN LAKSHMI NAGAR, KOLAPAKKAM	DOCUMENT NO				REV	0
		A23015-C-RB-01-DC-0002					
TITLE:	DESIGN CALCULATIONS	DESIGNED		CHECKED		APPROVED	
CLIENT:	M/S BUILD TECH ASSOCIATES	JSAN		NKK		NKK	
		DATE	02.08.2024	DATE	02.08.2024	DATE	02.08.2024

1.INTRODUCTION

Stilt +4 story Residential building has plan dimensions as per Floor plans.The building is located in seismic Zone- III on a site with Medium soil. The design wind speed is 50 m /sec for Chennai region. Design the building for seismic and wind loads as per IS: 1893(part-1)-2016 and IS: 875 (Part-3) - 2015.

2.STRUCTURAL SYSTEM

Conventional (Cast in-situ) RCC Framed structural system has been considered for the building.3-D Analysis has per been carried out in STAAD Pro.

3.MATERIALS

The Self weight of various elements are computed based on the Unit weight of materials as given below as IS:875 (Part-1) - 1987.

Table:1.Unit weight of materials(kN/m^3)

Steel	78.50
Plain cement concrete/Screed	24.00
Reinforced cement concrete(Dry)	25.00
Cement concrete solid block work	20.00
Water	9.81
Brick coba	18.00

3.1.Concrete

Grade of Concrete for all structural items (Cast in-situ) considered as M25.

3.1.1.Exposure Category

As per the clause 8.2.2 of IS:456 -2000, the exposure category considered is moderate refer table-2.

Table:2.Nominal cover to reinforcement

(Environmental exposure condition)

Structural element	Minimum(As per is456-2000)	Considered for design
Footing	50	50
Column	40	40
Beam	30	30
Slab	20	20

3.2 Reinforcement

Reinforcement steel shall be of Grade Fe500 confirming to IS:1786 – 2008.

3.3.Structural steel

All Structural steel shall conform to IS: 2062 -2011. Pipes and tubes shall conform to IS: 1161- 2014.

4.LOADS:

This load comprises of self - weight of all the elements in the structure. Additional weight of the elements which are not declared in the model are calculated as below and applied.

4.1.Dead loads(IS875-PART1-1987)

i)Wall load

Main wall load:

Wall thickness = 230 mm

Height of wall = 3000 mm

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Self weight of wall	=	Wall thickness * Height *Unit weight of Brick			
	=	0.23 x 3 x 20	=	13.8	kN/m
Partition wall load:					
Wall thickness			=	115	mm
Height of wall			=	3000	mm
Self weight of wall	=	Wall thickness * Height *Unit weight of Brick			
	=	0.115 x 3 x 20	=	6.9	kN/m
ii)Slab load:					
Floor/Roof slab Load:					
By self weight:					
Thickness of slab			=	125	mm
Self weight of slab	=	Thickness of slab * Unit weight of RCC			
	=	0.125 x 25	=	3.125	kN/m^2
By Finishing:					
Thickness of finishing			=	75	mm
Self weight of slab	=	Thickness of slab * Unit weight of PCC			
	=	0.075 x 24	=	1.8	kN/m^2
Total load	=	3.125 + 1.8	=	4.925	kN/m^2
Sunken slab Load:					
Thickness of slab			=	300	mm
Self weight of slab	=	Thickness of slab * Unit weight of PCC			
	=	0.3 x 24	=	7.2	kN/m^2
4.2.Live load(IS875-PART2-1987)					
Load on office/Tailoring eqip. room			=	5	kN/m^2
Load on Bathroom/balcony/staircase			=	3	kN/m^2
Load on roof			=	1.5	kN/m^2
4.3.Earthquake load(IS1893-PART1-2016)					
Length of building			=	24.39	m
Width of building			=	9.16	m
Slab thickness			=	0.125	m
Wall thickness			=	0.23	m
Zone location			=	Chennai	
Zone			=	III	
Total height from founding			=	12	m
Design horizontal acceleration Spectrum					
$A_h = \frac{Z I S_a}{2 R g}$					
Zone factor Z			=	0.16	
Importance factor I			=	1.5	
Response reduction factor R			=	5	
T_a = RC MRF building	=	$0.075\ h^{0.75}$	=	0.4836	
RC-STEEL composite MRF building	=	$0.080\ h^{0.75}$	=	0.5158	

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Steel MRF building = $0.085\ h^{0.75}$ = 0.548

all other buildings = $\frac{0.075\ h}{\sqrt{d}}$ = 0.2974

Type of building = RC MRF building

T_a = 0.48 seconds

$\frac{S_a}{g}$ = 0.42

Table.3)Design Acceleration co-efficient S_a/g

Rocky Hard soil	1+15T	T < 0.10 S
	2.5	0.10 s < T < 0.40 s
	1/T	0.40 s < T <4.0 s
	0.25	T > 4.00 s
Medium stiff soil	1+15T	T < 0.10 S
	2.5	0.10 s < T < 0.55 s
	1.36/T	0.55 s < T <4.0 s
	0.34	T > 4.00 s
Soft soil	1+15T	T < 0.10 S
	2.5	0.10 s < T < 0.67 s
	1.67/T	0.67 s < T <4.0 s
	0.42	T > 4.00 s

$A_h = \frac{0.16 \times 1.5 \times 0.42}{2 \times 5}$ = 0.0101

Design base shear VB will be evaluated by STAAD analysis is considering 100% Dead load and 25% of the live load .

4.4.Wind load(IS875-PART 3-2015)

Building location		=	Chennai	
Basic Wind Speed	V_b	=	50	m / sec
Greater horizontal dimension of the building	l	=	27.8	m
Lesser horizontal dimension of the building	w	=	12.7	m
Maximum height of the building	z / h	=	12.5	m
Design life of structure		=	50	Years,
Terrain Category		=	I	
Terrain slope q		=	0	°
Type of structure		=	All other building	
Type of building		=	Non Circular	
Maximum Frame (Bay) spacing		=	3.75	m
Roof angle		=	0	°

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Design Calculation:

Design wind speed, $V_z = [V]_b \, k_1 \, k$ Where,

V_z = Design Wind Speed at height z, m/sec

k_1 = Probability factor (risk coefficient) (Ref: Cl:6.3.1, Table-1)

k_2 = Terrain roughness and height factor (Ref. Cl:6.3.2)

k_3 = Topography factor (Ref. Cl:6.3.3)

k_4 = Importance factor for the cyclonic region (Ref. Cl:6.3.4)

,Shall be taken 1.15 where the buidling within 200km from the coast (Ref: Cl: 6.6)

k_1 = 1

k_2 = 1

k_3 = 1

k_4 = 1

V_z = 50 x 1 x 1 x 1 = 50 m/s

Calculate design wind pressure,

Wind pressure, $P_z = [\![0.6\ V]\!]$ Ref: Cl:7.2

Where,

P_z = Wind pressure at height, z in N/m^2

V_z = Design wind speed at height, z in m/sec

P_z = 0.6 x 50 x 50 = 1500 N/m^2

Design wind pressure,

$$P_d = K_d K_a K_c P_z \geq [\![$$

Where,

P_d = Design wind pressure, N/m^2 Ref : Cl:7.2

K_d = Wind directionality factor Ref : Cl:7.2.1

(For Local pressure coefficient & cyclone affected region, Kd shall be 1.0 with respect to any shape)

K_a = Area averaging factor Ref : Cl7.2.2

K_c = Combination factor Ref : Cl:7.3.3.13

P_z = Wind pressure at height, N/m^2 Ref : Cl:7.2

K_d = 0.9

K_a = 0.8

K_c = 0.9

Tributary Area, = 353.06 Sqm

P_{d1} = 1500 x 0.9 x 0.8 x 0.9 = 972 kN/m^2

P_{d2} = 0.7 x 1500 = 1050 kN/m^2

P_d = = 1050 kN/m^2

5.LOAD COMBINATIONS:

The following load combinations are considered in scheme and detailed design stages in reference with IS 456:2000, IS 1893 (Part1): 2016 & IS 875 (Part5)

Load combinatoons used for foundation design

DL+SIDL+LL

DL+SIDL+LL±WL_X

DL+SIDL+LL±WL_Z

DL+SIDL+LL+EQ

DL+SIDL±WL_X

DL+SIDL±WL_Z

DL+SIDL±EQ

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Table:4 Load combinations used for scheme and detail designs:(Limit state method)

Serviceability	Strength
DL+SIDL+LL+RLL	1.5DL+1.5SIDL+1.5LL+1.5RLL
DL+SIDL+0.8LL±0.8WL_X	1.5DL+1.5SIDL+1.5LL±1.5WL_X
DL+SIDL+0.8LL±0.8WL_Z	1.5DL+1.5SIDL+1.5LL±1.5WL_Z
DL+SIDL+0.8LL±0.8EQ	1.5DL+1.5SIDL+1.5LL±1.5EQ
DL+SIDL+±0.8WL_X	0.9DL+0.9SIDL±1.5WL_X
DL+SIDLL±0.8WL_Z	0.9DL+0.9SIDL±1.5WL_Z
DL+SIDLL±0.8EQ	0.9DL+0.9SIDL±1.5EQ
	1.2DL+1.2SIDL+1.2LL±1.2WL_X
	1.2DL+1.2SIDL+1.2LL±1.2WL_Z
	1.2DL+1.2SIDL+1.2LL±1.2EQ

5.1.Deflection criteria:

Permissible deflections: IS 456:2000 (Clause 23.2 (b))

Max vertical deflection on beams and slab under vertical load:

Final deflection	≤	span/250	IS 456:2000 (Clause 23.2 (a))
Final deflection after erection of all loads&finishes	≤	span/350 (or)20mm whichever is less.	

Max storey drift due to lateral load:

Under wind load	≤	Height/500	IS 456:2000 (Clause 20.5)
Under earthquake load	≤	Height/350	IS 1893, part1 :2016 (Clause 7.11.1.1)

Long term deflections due to shrinkage and creep will be calculated as per Annexure-C of IS 456:2000.

5.2.Crack width requirements:

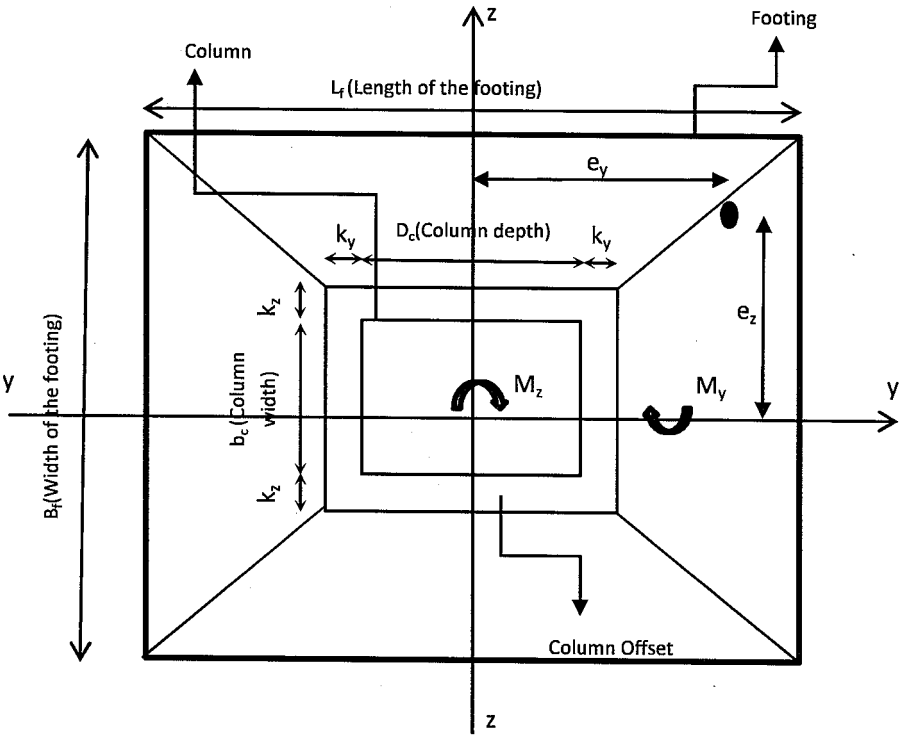
Crack width for all structural members shall be limited to 0.3 mm.

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INPUT:

Founding Depth	=	2	m
Safe bearing capacity of soil(SBC)	=	207	kN/m²
Unit weight of soil	=	18	kN/m²
Unit weight of concrete	=	25	kN/m²
Bearing capacity factor(BCF)	=	1	
Cover to reniforcement	=	50	mm
Eccentricity along z-z direction(e_z)	=	0	mm
Eccentricity along y-y direction(e_y)	=	0	mm
Offset along z-z direction(k_z)	=	50	mm
Offset along y-y direction(k_y)	=	50	mm
Type of footing		Rectangular/Sloped	



Plan of Footing

Column Numeration Details:-

Depth of column (Dc)mm	Width of coulum(bc)mm
450	230

Grade of Concrete /Rft

Concrete (F_{ck}) N/mm²	Steel (F_y) N/mm²
25	500

Footing Numeration Details:-(Units in mm)

Length (L_f)	Width (B_f)	Depth @ column face (D)	Depth @ toe (d)	Effective depth (d_{eff}) mm
2250	2250	600	300	545

Size of Reinforcement

Location	Along y-y(longer bar)	Along z-z(shorter bar)
Bottom	10	10
Top	8	8

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Stadd-Pro Output

IS 456-2000

ANALYSIS RESULT:

Moment about z-z M_z	=	5	kNm	Clockwise
Moment about y-y M_y	=	5	kNm	Clockwise
Axial Load (P)	=	850	kN	Compression
Load Factor	=	1.5		

Moment due to Eccentricity:

About z-z = Axial load x Eccentricity along y-y
= 850 x 0 = 0 kNm

About y-y = Axial load x Eccentricity along z-z
= 850 x 0 = 0 kNm

TOTAL MOMEMT:

About z-z(M_z)	=	5	kNm	Clockwise
About y-y(M_y)	=	5	kNm	Clockwise

Gross Bearing Capacity:

GSBC = SBC x BCF+Founding depth x Unit weight of soil
= 207 x 1 + 2 x 18 = 243 kN/m²

$$P = \frac{P}{A} \pm \frac{M_z}{Z_z} \pm \frac{M_y}{Z_y} \leq SBC$$

$$P1 = \frac{P}{A} + \frac{M_z}{Z_z} + \frac{M_y}{Z_y} = \frac{850}{2.25 \times 2.25} + \frac{5 \times 6}{2.25 \times 2.25 \times 2.25} + \frac{5 \times 6}{2.25 \times 2.25 \times 2.25} = 173 \text{ kN/m}^2$$

$$P2 = \frac{P}{A} + \frac{M_z}{Z_z} - \frac{M_y}{Z_y} = \frac{850}{2.25 \times 2.25} + \frac{5 \times 6}{2.25 \times 2.25 \times 2.25} - \frac{5 \times 6}{2.25 \times 2.25 \times 2.25} = 168 \text{ kN/m}^2$$

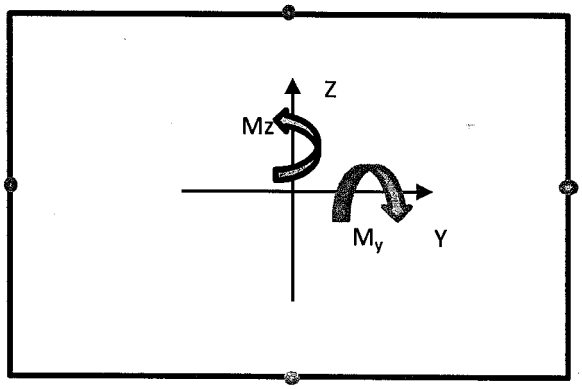
$$P3 = \frac{P}{A} - \frac{M_z}{Z_z} - \frac{M_y}{Z_y} = \frac{850}{2.25 \times 2.25} - \frac{5 \times 6}{2.25 \times 2.25 \times 2.25} - \frac{5 \times 6}{2.25 \times 2.25 \times 2.25} = 163 \text{ kN/m}^2$$

$$P4 = \frac{P}{A} - \frac{M_z}{Z_z} + \frac{M_y}{Z_y} = \frac{850}{2.25 \times 2.25} - \frac{5 \times 6}{2.25 \times 2.25 \times 2.25} + \frac{5 \times 6}{2.25 \times 2.25 \times 2.25} = 168 \text{ kN/m}^2$$

Safe

P5 168 kN/m²

P3 163 kN/m² P1 173.17 kN/m²



P7 165 kN/m² P8 171 kN/m²

P4 168 kN/m² P6 168 kN/m² P2 168 kN/m²

Base Pressure Below the Footing

FLEXURAL REINFORCEMENT

Bottom Reinforcement

Reinforcement along Longer Direction (Along y-y)

i)Factored moment @ longer direction per "m" width(to the centre of column)

Factored Moment due to P3,P5,P1 Mz

$$= \frac{[MAX(P3+P5),(P5+P1)]}{2} \times \frac{((L_f - D_c)*0.5)^2)}{2} \times 1.5$$

$$= \frac{167.92+173.18}{2} \times \frac{((2250-450)*0.5)^2}{2} \times 1.5 = 103.62 \text{ kNm}$$

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Factored moment due to P4,P6,P2 Mz $= \frac{[MAX(P4+P6),(P6+P2)]}{2} \times \frac{((L_f-D_c)*0.5)^2)}{2} \times 1.5$

$= \frac{167.92+167.92}{2} \times \frac{((2250-450)*0.5)^2}{2} \times 1.5 =$ 102.02 kNm

Check the footing depth for BM

$d = \sqrt{\frac{\text{Max moment}}{0.133 Fck\ b}}$

$= \sqrt{\frac{103.62x10^6}{0.133 \times 25 \times 1000}} =$ 176.533 mm

< 545 mm

Safe

$A_{st.req} = \frac{bd*f_{ck}}{2f_y}[1-\sqrt{1-\frac{4.598M_u}{F_{ck}\,bd^2}}]$

$= \frac{2250x25x545}{2 \times 500}[1-\sqrt{1-\frac{4.598 \times 176.6 \times 10^6}{25 \times 2250 \times 545^2}}] =$ 440.27 mm²

$A_{st.min}$ = 0.12% of bD $= \frac{0.12}{100} \times 1000 \times 600 =$ 720 mm²

$P_{t.required} = \frac{Max(A_{st.req},A_{st.min})}{bd_{eff}} \times 100 = \frac{720}{1000 \times 545} \times 100 =$ 0.13%

Spacing Required $= \frac{\pi \times 10^2}{4 \times 720} \times 1000 =$ 109 mm

Spacing Provided = 105 mm

Safe

$P_{t.\ Provided} = \frac{\pi \times 10^2}{4 \times 105 \times 545} \times 100 =$ 0.14% > 0.13%

Safe

Provide T10 mm dia of reinforcement @ 105 mm c/c

Reinforcement along shorter Direction (Along z-z)

i)Factored moment@shorter direction per "m" width(to the centre of column)

Factored Moment due to P4,P3,P7 My $= \frac{[MAX(P3+P7),(P7+P4)]}{2} \times \frac{((B_f-b_c)*0.5)^{\wedge 2})}{2} \times 1.5$

$= \frac{165.28+167.92}{2} \times \frac{((2250-230)*0.5)^2}{2} \times 1.5 =$ 127.46 kNm

Factored Moment due to P2,P1,P8 My $= \frac{[MAX(P2+P8),(P8+P1)]}{2} \times \frac{((B_f-b_c)*0.5)^{\wedge 2})}{2} \times 1.5$

$= \frac{173.18+170.54}{2} \times \frac{((2250-230)*0.5)^2}{2} \times 1.5 =$ 132 kNm

Checking of footing depth for Factored BM

$d = \sqrt{\frac{\text{Max moment}}{0.133 Fck\ b}}$

$= \sqrt{\frac{131.5x10^6}{0.133 \times 25 \times 1000}} =$ 198.869 mm

Safe

Ast.req $= \frac{2250x545x25}{2 \times 500}[1-\sqrt{1-\frac{4.598 \times 131.6 \times 10^6}{25 \times 2250 \times 545^2}}] =$ 559.83 mm²

$A_{st.min}$ = 0.12% of bD $= \frac{0.12}{100} \times 1000 \times 600 =$ 720 mm²

$P_{t.required} = \frac{Max(A_{st.req},A_{st.min})}{bd_{eff}} \times 100 = \frac{720}{1000 \times 545} \times 100 =$ 0.13%

Spacing Required $= \frac{\pi \times 10^2}{4 \times 720} \times 1000 =$ 109 mm

Spacing Provided = 105 mm

Safe

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		DATE	02.08.2024	DATE	02.08.2024	DATE	02.08.2024

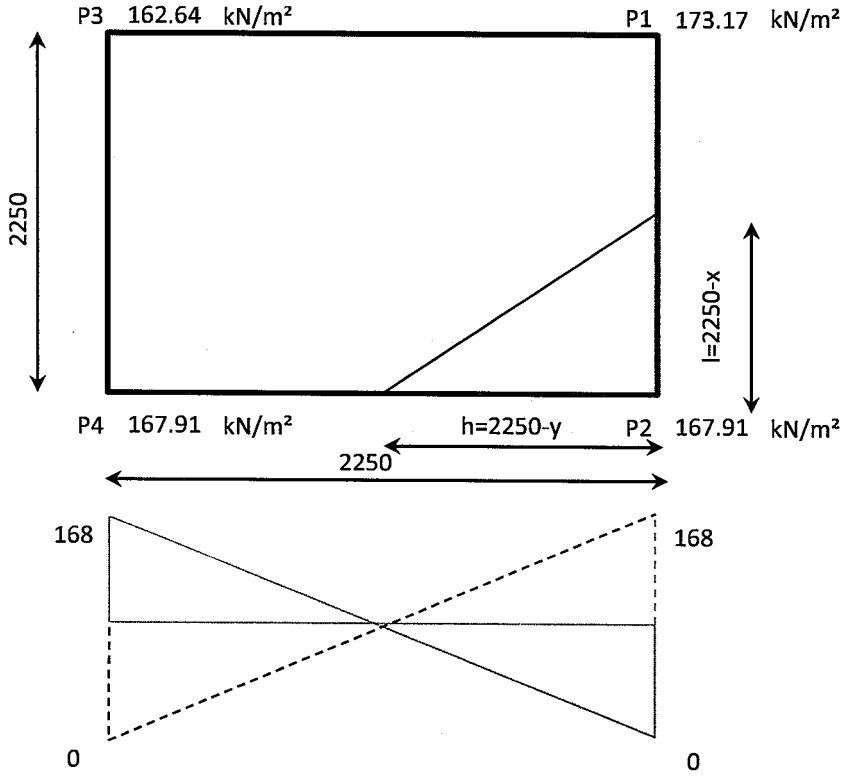
$$P_{t.\ Provided} = \frac{\pi \times 10^{\wedge}2}{4 \times 105 \times 545} \times 100 = 0.14\% > 0.13\%$$

Safe

Provide T10 mm dia bars @105 mm Spacing along shorter direction

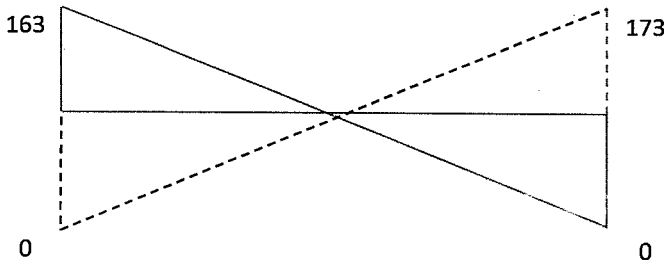
Top Reinforcement

Total area of footing(A_f) = 5.06 m^2

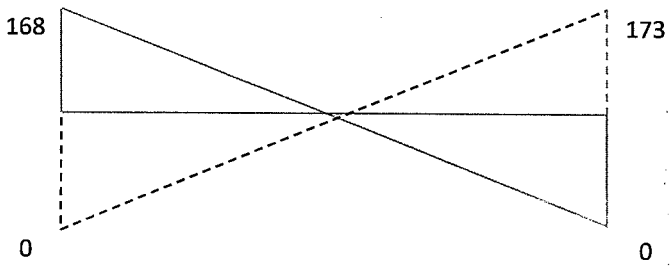


By P4,P2 $\frac{y}{167.91} = \frac{2250-y}{0}$

y = 2250 h = L_f-y = 0 mm



By P3,P1 y = 2250 l = L_f-y = 0 mm

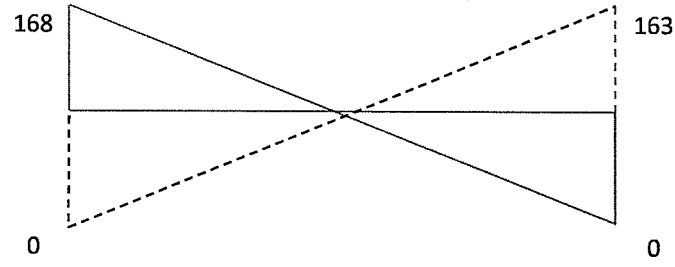


By P2,P1

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x = 2250 l = B_f-x = 0 mm



By ,

x = 2250 l = B_f-x = 0 mm

Area of tension zone (Loss of area) $= \frac{1}{2} \times lh$

$= \frac{1}{2}(0x0 + 0x0)$ = 0.00 m^2

Area of compression zone = Total area-loss of area = 5.06

Loss of contact $= \frac{\text{Loss area}}{\text{Total area}} \times 100 = \frac{0.00}{5.06} \times 100$ = 0 %

< 10 %

Size of footing is safe

Reinforcement along y-y (Applicable When the Negative pressure is generated)

i)Factored Moment @ longer direction per "m" width(to the centre of column)

Factored Moment due to P3,P5,P1 Mz $= \frac{\text{No (-ve) Pressure}}{2} \times \frac{((L_f-D_c)*0.5)^{\wedge}2 \times 1.5}{2}$

$= \frac{\text{Not applicable}}{2} \times \frac{((2250-450)*0.5)^{\wedge}2}{2} \times 1.5$ = 0 kNm

Factored Moment due to P4,P6,P2 Mz $= \frac{\text{No (-ve) Pressure}}{2} \times \frac{((L_f-D_c)*0.5)^{\wedge}2 \times 1.5}{2}$

$= \frac{\text{Not applicable}}{2} \times \frac{((2250-450)*0.5)^{\wedge}2}{2} \times 1.5$ = 0 kNm

$A_{st.req} = \frac{1000x545x25}{2 \times 500}\left[1-\sqrt{1-\frac{4.598\ x0 \times 10^{\wedge}6}{25 \times 1000 \times 545^{\wedge}2}}\right]$ = 0 mm^2

$A_{st.min}$ = 0.06% of bD $= \frac{0.06}{100} \times 1000 \times 600$ = 360 mm^2

$P_{t.required} = \frac{Max(A_{st.req} x A_{st.min})}{bd_{eff}} \times 100 = \frac{0}{1000 \times 545} \times 100$ = 0.00%

Spacing Required $= \frac{\pi \times 8^{\wedge}2}{4 \times 360} \times 1000$ = 125 mm

Spacing Provided = 125 mm Safe

$P_{t.\ Provided} = \frac{\pi \times 8^{\wedge}2}{4 \times 125(600-50)} \times 100$ = 0.08% > 0.06%

Safe

Top reinfocrement is not required in longer direction

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Reinforcement along z-z

i)Factored moment @ Shorter direction per "m" width(to the centre of column)

Factored Moment due to P4,P3,P7 My $= \frac{\text{No (-ve) Pressure}}{2} \times \frac{((B_f-b_c)*0.5)^{\wedge}2 \times 1.5}{2}$

$= \frac{\text{Not applicable}}{2} \times \frac{((2250-230)*0.5)^{\wedge}2}{2} =$ 0 kNm

Factored Moment due to P2,P1,P8 My $= \frac{\text{No (-ve) Pressure}}{2} \times \frac{((B_f-b_c)*0.5)^{\wedge}2 \times 1.5}{2} =$

$= \frac{\text{Not applicable}}{2} \times \frac{((2250-230)*0.5)^{\wedge}2}{2} =$ 0 kNm

$A_{st.req} = \frac{1000 \times 545 \times 25}{2 \times 500}\left[1-\sqrt{1-\frac{4.598 \times 0 \times 10^{\wedge}6}{25 \times 1000 \times 545^{\wedge}2}}\right] =$ 0 mm^2

$A_{st.min}$ = 0.06% of bD $= \frac{0.06}{100} \times 1000 \times 600 =$ 360 mm^2

$P_{t.required} = \frac{Max(A_{st.req}, A_{st.min})}{bd_{eff}} \times 100 = \frac{0}{1000 \times 545} \times 100 =$ 0.00%

Spacing Required $= \frac{\pi \times 8^{\wedge}2}{4 \times 360} \times 1000 =$ 125 mm

Spacing Provided = 125 mm Safe

$P_{t.\ Provided} = \frac{\pi \times 8^{\wedge}2}{4 \times 125 \times 545} \times 100 =$ 0.08% > 0.06%

Safe

Top reinfocrement is not required in shorter direction

SHEAR CALCULATION

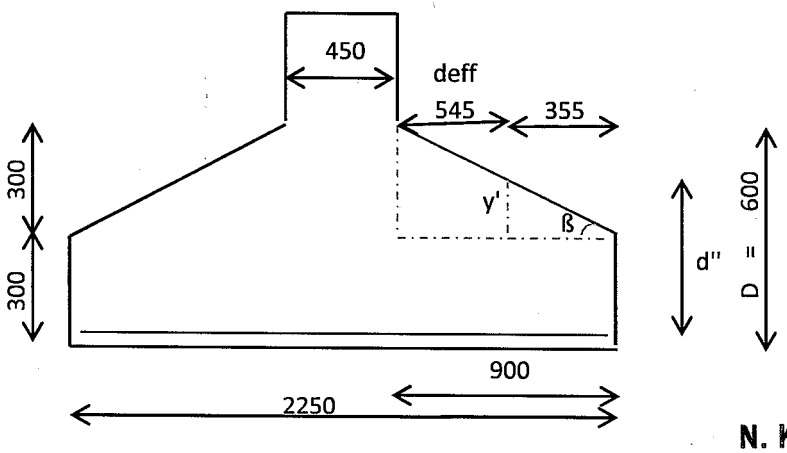
One way shear :

In Z direction

Shear force along Z direction by P5 , P6

V_{uy} = Max(P5 , P6)*Bf*(0.5*(Lf-Dc-2deff))*1.5

= 167.91 x 2250 (2250 - 450 - 2 x 545) x 0.5 x 1.5 = 202 kN



y' = 119 mm

d'' = 300 + 119 - 50 - 5 = 364 mm

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IS456-2000 P.NO.72

One way shear resistance :

$$\tau_v = \frac{V_{uy} - \frac{M_{uy} \tan ß}{d''}}{B_f * d''} \qquad \tan ß = \frac{300}{900} = 0.334$$

$$= \frac{202 \times 1000 - \frac{103.62 \times 10^6 \times 0.334}{364}}{2250 \times 364} = 0.13 \ N/mm^2$$

Design shear strength:

SP 16 CLAUSE 4.1

$$\tau_c = \frac{0.85\sqrt{(0.8\ Fck)} * (\sqrt{(1+5\beta)} - 1)}{6\beta}$$

$$pt = 100Ast/bd$$

(P.NO. 175)

$$\beta = 0.8Fck/6.89pt$$

$$= \frac{0.8 \times 25}{6.89 \times 0.14} = 21.15$$

$$\tau_c = \frac{0.85 \times \sqrt{(0.8 \times 25)} (\sqrt{(1+5 \times 21.15)} - 1))}{6 \times 21.15} = 0.28 \ N/mm^2$$

$\tau v < \tau_c$ Safe

In Y direction

Shear force along Y direction by P7 , P8

V_{uz} = Max(P7 , P8)*Lf*(0.5*(Bf-bc-2deff)) *1.5

= 170.54 x 2250 (2250 - 230 - 2 x 545) x 0.5 x 1.5 = 268 kN

y' = 139 mm

d'' = 300 + 139 - 50 - 5 = 384 mm

One way shear resistance:

IS456-2000 P.NO.72

$$\tau_v = \frac{V_{uz} - \frac{M_{uz} \tan ß}{d''}}{L_f * d''} \qquad \tan ß = \frac{300}{1010} = 0.298$$

$$= \frac{268 \times 1000 - \frac{131.5 \times 10^6 \times 0.298}{384}}{2250 \times 384} = 0.19 \ N/mm^2$$

Design shear strength :

$$\tau c = \frac{0.85\sqrt{(0.8\ Fck)} * (\sqrt{(1+5\beta)} - 1)}{6\beta} \quad , \quad pt = 100Ast/bd$$

(P.NO. 175)

$$\beta = 0.8F_{ck}/6.89p_t$$

$$= \frac{0.8x25}{6.89 \times 0.138} = 21.15$$

$$\tau c = \frac{0.85 \times \sqrt{(0.8x)} (\sqrt{(1+5 \times 21.15)} - 1))}{6 \times 21.15} = 0.28 \ N/mm^2$$

$\tau v < \tau_c$ Safe

Two way shear

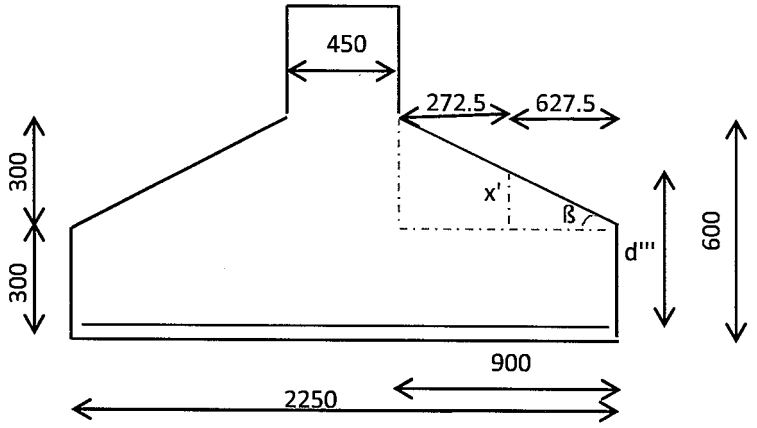
The critical section is located d/2 from the periphery of the column

Factored shear force V_{uz} = 1.5 x Pu

= 1.5 x 850 = 1275 kN

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x' = 210 mm

d''' = 300 + 210 - 50 - 5 = 455 mm

Two way

shear resistance V_c = Ks x Tc2 x d''' x 2 x [(D_c+deff)+(b_c+deff)] = 2013.4 kN

IS 456-2000

τ_{c2} = 0.25√fck = 0.25 x √25 = 1.25 N/mm²

K_s = 0.5+β_c < 1

$$\beta_c = \frac{\text{Short side of column}}{\text{Long side of column}} = \frac{230}{450} = 0.51$$

K_s = 0.5 + 0.52 < 1

= 1.01 > 1

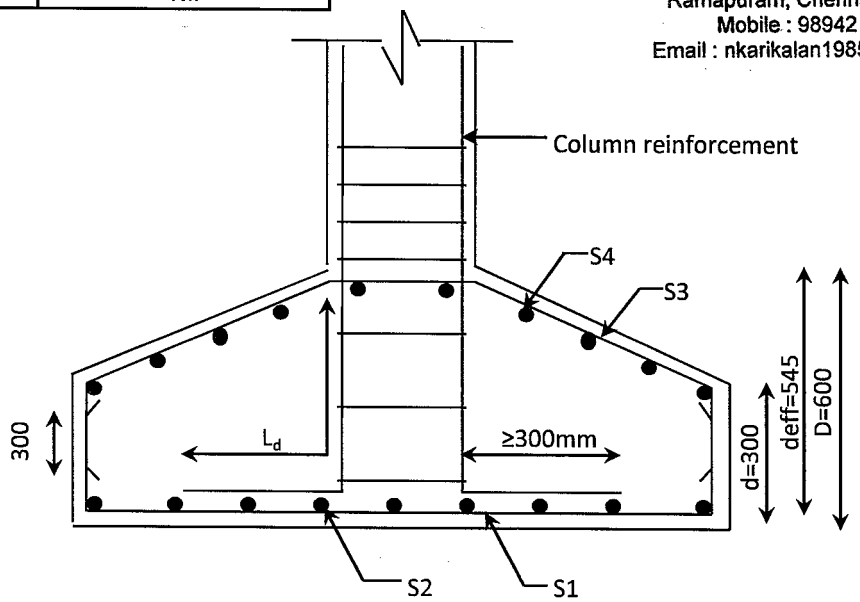
Take Ks as 1

V_{uz} < V_c SAFE

Schedule of RC Details:-

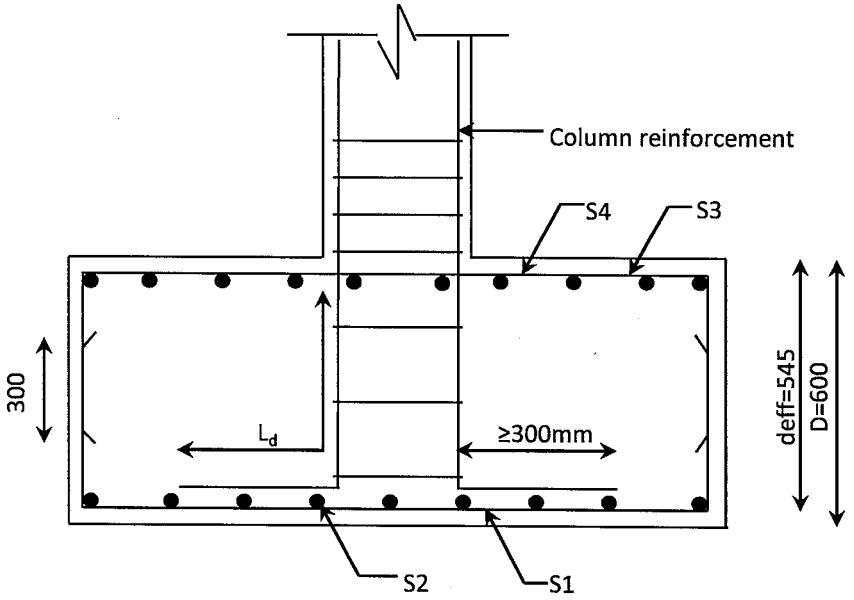
S1	T10 at 105 mm C/C
S2	T10 at 105 mm C/C
S3	Nil
S4	Nil

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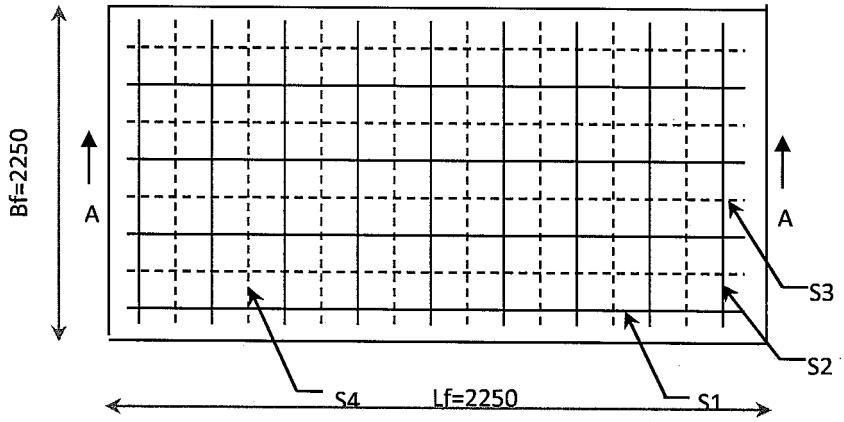


Section A-A-RC details of Trapezoidal Footing

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Section A-A-RC details of Flat Footing



Plan of RC Details

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Input:

Depth of column D)		=	380	mm
Width of column (D_z)		=	230	mm
Unsupported length (L_y)		=	3000	mm
L_z			3000	mm
Column condition against sidesway		=	Restrained	
Multiplication factor of (k_y,k_z)	k_{y1}	=	1.2	
	k_{z1}	=	1.2	
Effective length (L_{ey})		=	3600	
Effective length (L_{ez})		=	3600	
Clear cover(cc)		=	50	mm

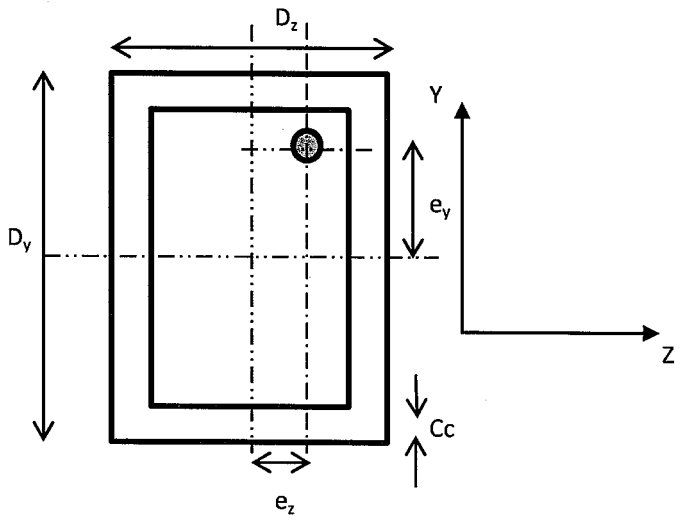


Fig:1 Cross section of column

Stadd.Pro

Analysis Result

Axial load (kN) P_u			1798	Compression
Moment (kNm)	M_{uy}	Bottom	39	Clock-wise
	M_{uz}	Bottom	-23	Anti-clockwise

Axial load - [+ve= Compression , -ve=Tension]

Moment -[+ve = Clock-wise, -ve = Anti-clockwise]

Grade of Materials

Concrete (F_{ck}) N/mm^2	Steel (F_y) N/mm^2
25	500

Reinforcement Details

Longitudinal bar(mm)	20
Tie bar(mm)	8
Reinforcement %	3

Slenderness Ratio:

$$\text{About y-y} = \frac{L_{ey}}{Dy} = \frac{3600}{380} = 9.4737 < 12$$

$$\text{About z-z} = \frac{L_{ez}}{Dz} = \frac{3600}{230} = 15.652 > 12$$

Long-Column

Column is slender about z-z axis

$$\text{Effective cover d'} = cc + \frac{\text{Dia of bar}}{2} = 50 + \frac{20}{2} = 60 \text{ mm}$$

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$$\frac{d'}{D} = \frac{60}{380} = 0.16$$

$$\frac{d'}{B} = \frac{60}{230} = 0.26$$

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Minimum eccentricity:

e_min =Max of (((Unsupported length)/500+(Late

$$e_{min.y} = \frac{3000}{500} + \frac{380}{30} = 18.67 < 20 = 20 \text{ mm}$$

$$e_{min.z} = \frac{3000}{500} + \frac{230}{30} = 13.67 < 20 = 20 \text{ mm}$$

Additional Moment Calculation:

M_a = P_(u) e_(y) k_y

M_a = Additional moment along major and minor axis

e_a = Additional eccentricity along major and minor axis

ky = Reduction factor

**e_a = *D_y/2000* x *(L_ey/*

$$e_{ay} = \frac{230}{2000} \times \left(\frac{3600}{230}\right)^{\wedge 2} = 28.174 \text{ mm}$$

$$e_{az} = \frac{380}{2000} \times \left(\frac{3600}{380}\right)^{\wedge 2} = 17.053 \text{ mm}$$

k = Min of((P_uz/P_uz - P_u/P

Here,

P_{uz} = Maximum pure compression strength of the column

P_b = Axial strength corresponding to balance failure about the axis

P_u = Axial load

$P_{uz} = 0.45 f_{ck} \times D \times b + (0.75 \times f_y - 0.45_{fck}) A_s$

$= (0.45 \times 380 \times 230) + (0.75 \times 500 - 0.45 \times 25)(0.03 \times 380 \times 230)$

$= 1937.01$ mm

IS456 1978 Table 6

P_b= (k_1+k_2 p/f_ck)f_ckBD

$k_{y1} = 0.194$

$k_{y2} = 0.222$

$P_{by} = (0.194 + 0.222 \times 0.03) \times 25 \times 380 \times 230 = 438.44$ kN

$k_y = \frac{1937.01}{1937.01} - \frac{1798}{438.44}, 1 = 0.0928$

$k_{z1} = 0.17$

$k_{z2} = -0.116$

$P_{by} = (0.17 + -0.116 \times 0.03) \times 25 \times 380 \times 230 = 363.85$ kN

$k_z = \frac{1937.01}{1937.01} - \frac{1798}{363.85}, 1 = 0.0884$

$M_{ay} = 1798 \times 28.1739130434783 \times 0.0928 = 4.7$ kNm

$M_{az} = 1798 \times 17.0526315789474 \times 0.0884 = 2.71$ kNm

Moment Due to Eccentricity:

$M_e = P_u \times e_{min}$

$M_{ey} = \frac{1798 \times 20}{1000} = 35.96$ kNm

$M_{ez} = \frac{1798 \times 20}{1000} = 35.96$ kNm

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Design moment:

$M_d = \text{Max}(M_e, M_u) + M_a$

$M_{dy} = \text{Max}(39, 35.96) + 4.7 = 43.7$ kNm

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M_{dy} = Max(23,35.96) + 2.71 = 38.67 kNm

Uniaxial moment capacity about y-y axis:

$\frac{P_u}{F_{ck} * BD} = \frac{1798 \times 10^3}{25 \times 230 \times 380}$ = 0.82

$\frac{M_{uy}}{F_{ck} * bD^2}$ = = 0.10095

$M_uy1 = M_uy/(f_ck$

= 0.10095 x 25 x 230 x 380^2 = 84 kNm

Uniaxial moment capacity abouz-z axis:

$\frac{P_u}{F_{ck} * BD} = \frac{1798 \times 10^3}{25 \times 230 \times 380}$ = 0.82

$\frac{M_{uz}}{F_{ck} * DB^2}$ = = 0.087

$M_uz1 = M_uz/(f_ck$

= 0.08675 x 25 x 380 x 230^2 = 43.596 kNm

Calculation of P_{uz}

p_t = 3 %

$\frac{P_u}{P_{uz}} = \frac{1798}{1937}$ = 0.9282

(P_u/P_{uz})	α^n
0.2	1
0.8	2
Other values	Interpolate

α^n = 2.22

$\frac{M_{dy}}{M_{uy1}} = \frac{43.7}{84}$ = 0.53

$\frac{M_{dz}}{M_{uz1}} = \frac{38.67}{43.5962}$ = 0.89

Criteria for Biaxial Bending (Bresler's Formulation)

IS 456-2000
Cl 39.6
P.NO:71

$[Mdy/Muy1]\alpha^n+[Mdz/$

(0.53) ^ 2.22 + (0.89) ^ 2.22 = 1.02 ≥ 1

Check for Maximum and Minimum Reinforcement

IS 456-2000
Cl 26.5.3.1
P.No: 48

Maximum reinforcement = 4% of cross sectional area

$= \frac{4}{100}$ x 380 x 230 = 3496 mm^2

Minimum reinforcement = 0.8% of cross sectional area

$= \frac{0.8}{100}$ x 380 x 230 = 699.2 mm^2

Provided reinforcement = 3% of cross sectional area

$= \frac{3}{100}$ x 380 x 230 = 2622 mm^2

Codal provisions are satisfied

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PROJECT:	PROPOSED RESIDENTIAL BUILDING AT VGN LAKSHMI NAGAR, KOLAPAKKAM	DOCUMENT NO				REV	0
		A23015-C-RB-01-DC-0002					
TITLE:	DESIGN OF COLUMN	DESIGNED		CHECKED		APPROVED	
CLIENT:	M/S BUILD TECH ASSOCIATES	JSAN		NKK		NKK	
		DATE	02.08.2024	DATE	02.08.2024	DATE	02.08.2024

IS 456-2000

Clause 26.5.3.2

P.NO:49

Transverse reinforcement:

Lateral Ties

1) Dia of tie bar sholuld be

$$> \frac{\text{Maximum longitudinal dia of bar}}{4} = \frac{20}{4} = 5 \text{ mm}$$

> 6mm = 6 mm

Take high value 8 mm

2) Pitch

= Least dimesnion fo compression member = 230 mm

(or)

= 16*smallest dia of longitudinal bar = 320 mm

(or)

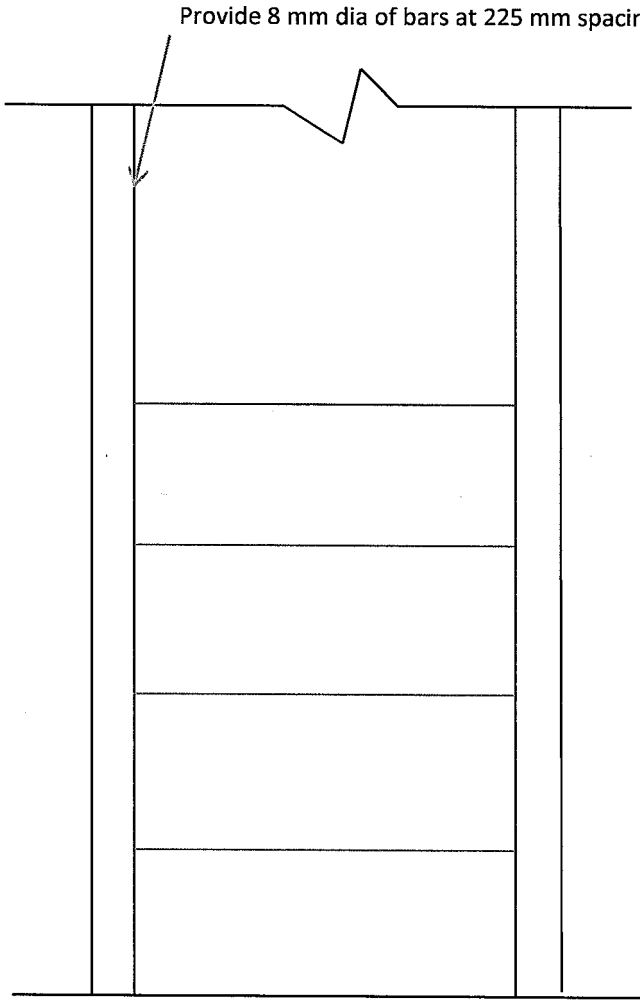
= 300mm = 300 mm

Take low value 225 mm

Provide 8 mm dia of bars at 225 mm spacing

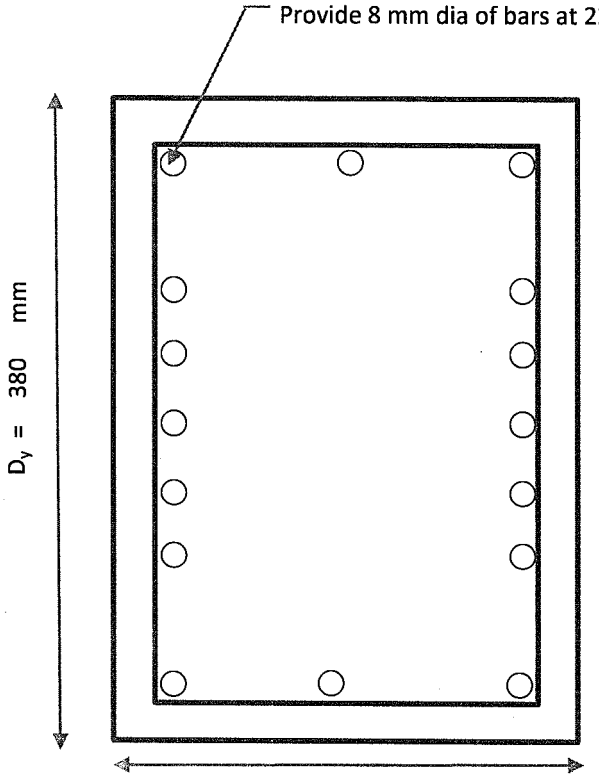
Provide closer spacing on either side of the column

àbove and below beam / slab.



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Stadd.pro

IS456-2000
Annex-G

Span(L)		=	3.1	m
Clear cover(C_C)		=	30	mm
Modulus of elasticity of concrete (Ec)	= $5000\sqrt{f_{ck}}$ (N/mm^2)	=	25000	
Modulus of elasticity of steel(E_s)	(N/mm^2)	=	200000	
Type of beam		=	Fixed	
Environment condition		=	Not Aggressive	

Beam Numeration Details:-

Depth of Beam (D)mm	Width of Beam(b)mm
380	230

Grade of Concrete /Rft

Concrete(F_{ck}) N/mm^2	Steel (F_y) N/mm^2
25	500

Longitudinal Reinforcement Details:

Location	Left Support		Mid		Right Support	
	Dia(mm)	No	Dia(mm)	No	Dia(mm)	No
Top	20	3	20	3	20	3
Bottom	20	3	20	3	20	2

Transverse Reinforcement Details:

Location	Left Support	Mid	Right Support
Bar dia	8	8	8
No of Leg	2	2	2

ANALYSIS RESULT:

Location	Left Support	Mid	Right Support
BM at Top (kNm)	192	0	198
BM at Bottom (kNm)	0	180	0
SF (kN)	200	15	215
Torsion (kNm)	0	0	0

DESIGN CALCULATIONS:(For Top BM at Left support)

Equiavelent Bending moment:-

$M_{el} = M_u + M_t$

$M_1 = T_u \frac{(1+D/b)}{1.7} = 0\left(\frac{1+ 380/230}{1.7}\right)$ = 0 kN.m

$M_{el} = 192 + 0$ = 192 kN.m

Limiting Moment:

M_u limit = 0.133*Fck*b*d*d

eff.depth = D-dia of bar x 0.5-cover = 340 mm

M_u limit = 0.133 x 25 x 23088.41 x 340 = 88.41 kNm

The section is doubly reinforced

$M_u > M_u$limit

Reinforcement At Tension Zone:

Singly Reinforced Beam:

$$A_(st.required) = (bdf_ck)/[\![2f]\!]_$$

$$= \frac{1000 \times 380 \times 25}{2 \times 500}\left[1-\left[1-\frac{4.598 \times 192 \times 10\text{^}6}{25 \times 1000 \times 340\text{^}2}\right]^{½}\right] = 0 \text{ mm}^2$$

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Doubly Reinforced Beam:

SP.16 / P.NO 10

$A_{st\ Req} = A_{st.limit} + \Delta A_{st}$

$A_{st.limit} = \frac{p_t\ limit}{100} \times bd$

$P_{t.limit} = \frac{0.36\ F_{ck} * b * X_{umax}}{0.87 F_y * b * d} \times 100$

$\frac{Xu_{max}}{d} = \frac{0.0035}{0.0055 + \frac{0.87 * f_y}{E_s}} = 0.456$

$Pt_{limit} = \frac{0.36 \times 25 \times 230 \times 0.456 \times 340}{0.87 \times 500 \times 230 \times 340} \times 100 = 0.94\ \%$

$A_{st.limit} = \frac{0.94}{100} \times 230*340 = 735.1\ mm^2$

$\Delta A_{st} = \frac{M_u - M_u limit}{0.87 \times f_y \times (D-d')}$

$d' = 40\ mm$

$\Delta A_{st} = \frac{192 - 88.41}{0.87 \times 500 \times (340 - 40)} = 794\ mm^2$

$A_{st\ Req} = 735.08 + 794 = 1529\ mm^2$

Minimum reinforcement

IS456-2000: $= \frac{0.85 \times bd}{Fy} = 132.94\ mm^2$

IS13920 Cl.6.2.1: $= \frac{0.24 \times \sqrt{Fck} * bd}{Fy} = 187.68\ mm^2$

$A_{st\ pro} = 1530\ mm^2$

Maximum reinforcement:

IS456-2000: $A_{st\ max} = 0.04bD = 3496\ mm^2$

IS13920 Cl.6.2.2: $A_{st\ max} = 0.025bD = 2185\ mm^2$

$> A_{st\ pro}$ SAFE

Provide3No's of 20mm Dia bars

Reinforcement At Compression Zone:

$A_{sc} = \frac{0.87 \times f_y \times \Delta A_{st}}{F_{sc} - 0.447 \times f_{ck}}$

SP.16 / P.NO 6

$E_{sc} = \frac{0.0035 x (X_{umax} - d')}{X_{umax}}$

$X_{umax} = 0.456 \times d = 155.04$

$E_{sc} = \frac{0.0035 \times (155.04 - 40)}{155.04}$

$= 0.002597007$

$F_{sc} = 405.63\ N/mm^2$

$A_{sc} = \frac{0.87 \times 500 \times 794}{405.63 - 0.447 \times 30} = 880.6\ mm^2$

Min $A_{sc} = 187.68\ mm^2$

$A_{sc\ provided} = 881\ mm^2$

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Provide 3 No's of 20mm dia bars

Longitudinal reinforcement Details:

Top face:-

Zone	M_{eq}	M_{ulim}	Section	A_{streq}	$A_{st.pro}$	A_{screq}	$A_{st.pro}$	Status
Left	192	88.41	Doubly	1529	1529	875.4	875.4	SAFE
Mid	0	88.41	Singly	0	187.7	0	187.7	SAFE
Right	198	88.41	Doubly	1575	1575	926.1	926.1	SAFE

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Bottom face:-

Zone	M_{eq}	M_{ulim}	Section	A_{streq}	$A_{st.pro}$	A_{screq}	$A_{st.pro}$	Status
Left	0	88.41	Singly	0	187.7	0	187.7	SAFE
Mid	180	88.41	Doubly	1437	1437	774	774	SAFE
Right	0	88.41	Singly	0	187.7	0	187.7	SAFE

SHEAR STRESS CALCULATION

Equiavelent Shear force:-

$$V_e = V_u + \frac{1.6T_u}{b} = 200 + \frac{1.6 \times 0}{0.23} = 200 \text{ kN}$$

$$\text{Shear Stress } \tau_v = \frac{V_e}{bd} = \frac{200 \times 1000}{230 \times 340} = 2.558 \text{ N/mm}^2$$

$$\text{Design Shear Stress } \tau_c = \frac{0.85\sqrt{(0.8\,Fck)}*(\sqrt{(1+5\beta)}-1)}{6\beta}$$

$$pt = \frac{100Ast}{bD} = \frac{100 \times 1530}{380 \times 230} = 1.75057\ \%$$

$$\beta = 0.8f_{ck}/6.89pt$$

$$= \frac{0.8 \times 30}{6.89 \times 1.76} = 1.99$$

(P.NO. 175)

$$\tau_c = \frac{0.85 \times \sqrt{(0.8 \times 25)}(\sqrt{(1+5 \times 1.99)}-1))}{6 \times 1.99} = 0.735 \text{ N/mm}^2$$

IS456-2000 TABLE NO 20

$$\tau_{cmax} = 3.1 \text{ /mm}^2$$

$$\tau_c < \tau_v < \tau_{cmax}$$

Provide shear reinforcement

$$V_{us} = Vu - \tau c*bd = 142.52 \text{ kN}$$

$$A_{sv} = 101 \text{ mm}^2$$

$$S_v = \frac{0.87 F_y A_{sv} d}{V_{us}} = \frac{0.87 \times 500 \times 101 \times 340}{142.52 \times 1000} = 104.813 \text{ mm}$$

Minimumm and maximum spacing of striuups:

As per IS13920-2016:

At support:

IS13920-2016 CL.6.3.5

Maximum Spacing is minimum of

i) $\frac{d}{4} = \frac{340}{4} = 85$ mm

ii) $8\Phi = 8 \times 20 = 160$ mm

(Take diametre of smallest bar)

iii) 100mm = 100 mm

Take spacing as per IS3920 = 85 mm

At Mid:

Maximum Spacing is minimum of

i) $\frac{d}{2} = \frac{340}{2} = 170$ mm

As per IS456-2000:

Maximum:

IS456-2000 CL.26.5.1.5

i) Max spacing of stirrups = 0.75d = 255 mm

ii) 300mm = 300 mm

Minimum:

$$A_{svmin} = \frac{0.4\ b\ S_v}{0.87\ F_y}$$

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$$S_v = \frac{(0.87 \times 500 \times 101)}{0.4 \times 230} = 477.6 \text{ mm}$$

S_{vprc} = 85 mm — SAFE

Provide T8 mm 2 Legged stirrups at 85 mm C/C

Transverse Reinforcement Details:

Zone	V_{eq}	τ_v	τ_c	τ_{cmax}	V_{us}	A_{sv}	S_v	Status
Left	200	2.5575	0.71914	3.1	143.8	101	85	SAFE
Mid	15	0.19182	0.71914	3.1	41.24	101	170	SAFE
Right	215	2.74936	0.74259	3.1	156.9	101	85	SAFE

DEVELOPMENT LENGTH:

$$L_d = \frac{\phi \sigma_s}{\tau_{bd}} = \frac{20 \times 0.87 \times 500}{3} = 2900 \text{ mm}$$

Bar diametre ϕ = 20 mm

Bond stress τ_{bd} = 3 N/mm²

DEFLECTION :

$$\delta_L = \frac{1}{384} \times \frac{wl^4}{EI_{eff}}$$

$$w_{service} = \frac{12M_{eq}}{l^{2 \times 1.5}} = \frac{12 \times 192}{3.1 \times 3.1 \times 1.5} = 159.834 \text{ kN}$$

$$I_{eff} = \frac{I_r}{1.2 - \frac{M_r}{M} \times \frac{Z}{d}\left(1-\frac{a}{d}\right) \times \frac{b_w}{b}}$$

$I_r \le I_{eff} \le I_{gr}$

Cracking moment $M_r = \frac{F_{cr} \times I_{gr}}{y_t} = \frac{3.5 \times 1051713334}{190} = 19.37$ kNm

y_t = 190 mm

I_{gr} = 1051713334 mm⁴

Moduulus of rupture $F_{cr} = 0.7\sqrt{F_{ck}}$ = 3.5 N/mm²

Modular ratio m = 8

$$\frac{b\,a^2}{2} = m\,Ast\,(d-a)$$

$$a^2 = \frac{2m\,Ast\,d - 2m\,Ast\,a}{b}$$

$$= \frac{8323200}{230} - \frac{24480}{230} * a$$

$$a^2 = 36188 - 106.434783\,a$$

Neutral axis a = 145 mm

Lever arm $Z = d - \frac{a}{3} = 340 - \frac{145}{3} = 331.7$ mm

Moment at service load = 128 kNm

$$I_r = \frac{ba^3}{12} + m\,A_{st.pro}\,(D-a)^2 + (m-1)\,Asc.pro\,(a-d')^2$$

$$I_r = \frac{230x145x145x145}{3} + 8 \times 1530 \times (380-145)^{\wedge}2$$

$$+ (8-1) \times 881 \times (145-40)^{\wedge}2$$

Critical moment of inertia I_r (mm^4) = 802377154

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$$I_{eff} = \frac{802377154.2}{1.2 - \frac{19.37}{128} \times \frac{331.7}{340}\left(1-\frac{145}{340}\right) \times \frac{230}{230}} = 719414248 \ mm^4$$

$Ir \le Ieff \le Igr$ $\quad I_{eff}\ (mm^4) = 802377155$

i)Deflection due to load

$$\delta_L = \frac{159.833506763788 \times 3.1 \times 3.1 \times 3.1 \times}{384 \times 25000 \times 802377155} = 1.92 \ mm$$

ii)Deflection due to shrinkage:

$$\delta_s = k_3 \times \Psi_{es} \times L^2$$

Shrinkage curvature $\Psi_{es} = k_4 \frac{\varepsilon_{cs}}{D} = 0.322 \frac{0.0003}{380} = 0.255 \times 10^{\wedge -6}$

$\varepsilon_{cs} = 0.0003$

if, $0.25 \le P_t - P_c < 1.0$ $\quad k_4 = \frac{0.72(P_t-P_c)}{\sqrt{p_t}} = \frac{0.72 \times 0.2}{0.4472136} \le 1 = 0.322$

if, $P_t - P_c \ge 1.0$ $\quad k_4 = \frac{0.65(P_t-P_c)}{\sqrt{p_t}} = \frac{0.65 \times 0.2}{0.4472136} \le 1 = 0.098$

$P_t - P_c = 1.76 - 1.01 = 0.2\ \%$

$k_4 = 0.322$

$k_3 = 0.125$

$\delta_s = 0.125 \times 0.255 \times 10\wedge -6 \times 3.1 \times 3.1 \times 10\wedge 6 = 0.30632 \ mm$

iii)Deflection due to creep:

$$\delta_c = \delta_{1c} - \delta_1$$

$$= \frac{5}{384} \times \frac{wl^4}{E_{cc} I_{eff}} - \frac{5}{384} \times \frac{wl^4}{E_c I_{eff}}$$

$$= \frac{5}{384} \times \frac{wl^4}{I_{eff}} \left(\frac{1}{Ecc} - \frac{1}{Ec}\right), \quad Ecc = \frac{Ec}{1+\theta}$$

$$= \frac{5}{384} \times \frac{wl^4}{I_{eff}} \theta \qquad \theta = 1.5$$

$$= \frac{159.833506763788 \times 3.1 \times 3.1 \times 3.1 \times}{384 \times 25000 \times 802377155} \times 1.5 = 2.88 \ mm$$

$\delta = \delta_l + \delta_s + \delta c = 5.106$ mm

Limiting Deflection:

IS 456-2000
CL 23.2

$$\delta_{max} = \frac{Span}{350}, \ 20$$

$= 8.857, \ 20$

$= 8.857$ mm

$\delta_{max} > \delta$ $\quad =$ SAFE

CRACK WIDTH CALCULATION

$$W_{cr} = \frac{3a_{cr} \times \varepsilon_m}{1 + \frac{2(a_{cr}-C_{min})}{h-x}}$$

$a_{cr} = [[0.5S]_2 + C^2_{min}]^{1/2}$

Cover to rft $C_{min} = 30$ mm

Spacing of bars $S = 100$ mm

$a_{cr} = 59$

$$\varepsilon_m = \varepsilon_1 - \frac{b(h-a)(g-a)}{3E_s A_s (d-a)}$$

$$\varepsilon_1 = \frac{Fs}{Es} \times \frac{(h-a)}{(g-a)}$$

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$$F_s = \frac{m*\text{Service moment}*(d-a)}{I_r}$$

$$= \frac{8x128x10^{\wedge}6\ (340-145)}{802377154.2} = 248.9 \quad N/mm^2$$

$D = h = 380$ mm

Crack width calculating distance from combression edje $= g = 20$ mm

$$\varepsilon_1 = \frac{248.9}{200000} \times \frac{(380-145)}{(20-145)} = -0.00234$$

$$\varepsilon_m = -0 - \frac{230(380-145)(20-145)}{3 \times 200000 \times 1530(340-145)} = -0.0023$$

$$W_{cr} = \frac{3 \times 59 \times -0.0023}{1 + \frac{2(59-30)}{(380-145)}} = -0.32651 \text{ mm}$$

IS456-2000 Limiting crack width = 0.3 mm

SAFE

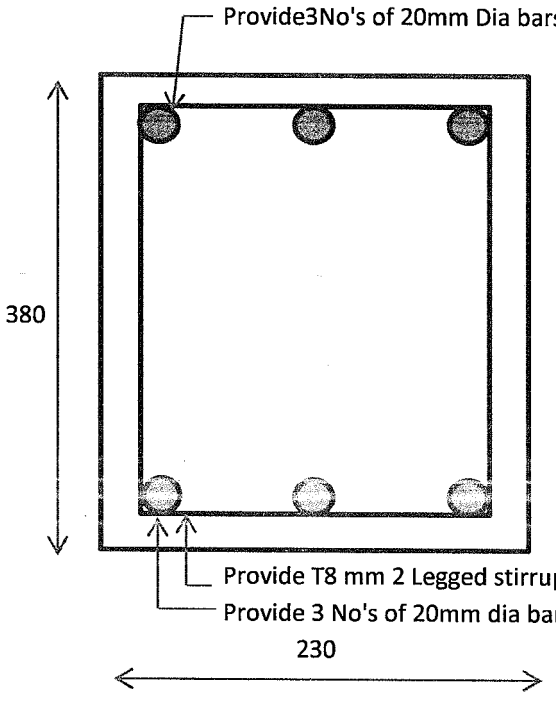


Fig.1 Cross section details

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CLIENT	M/S BUILD TECH ASSOCIATES	JSAN		NKK		NKK	
		DATE	02.08.2024	DATE	02.08.2024	DATE	02.08.2024

INPUT DETAILS:-

Grade of Concrete	F_{CK}	=	M25	
Grade of Steel	F_y	=	Fe500	
Cover to Main Reinforcement		=	20	mm
Thickness of the Slab		=	125	mm
Floor finishing thickness		=	20	
Assumed Dia of main Reinforcement		=	8	mm
Modulus of Elasticity OF steel	E_s	=	200000	N/mm^2
Support Condition		=	Fixed	
Creep Coefficient	θ	=	1.6	
crack width Distance	g	=	50	mm
Environment Condition		=	Not Aggressive	

SIZES OF PANEL:-

Clear Span	L_y	=	3.564	m	(Longer Span)
	L_x	=	3.072	m	(Shorter Span)
Effective depth	dx	=	101	mm	= (125-20-(0.5x8)
	dy	=	93	mm	= (101-8)
Effective Span (As per IS456:2000 Cl 22.2 Pg34)	L_y	=	3.657	m	= (3.564+93)
	L_x	=	3.173	m	= (3.072+101)

SPAN RATIO:-

$$r = \frac{Ly}{Lx} = 1 < 2$$

Hence The Slab is Two Way Slab

LOADS ON SLAB:-

Self weight of Slab	= (0.125x25)	=	3.125	N/m^2
Floor Finish	= (0.02x24)	=	0.48	N/m^2
Super imposed dead load	=	=	2	
Live Load		=	5	N/m^2
Factored total load	w	=	15.908	N/m^2

Boundary Condition — Two adjacent edges discontinuous

REINFORCEMENT CALCULATION:

Moment Cofficient				Bending Moment in kN-m			
Short Span		Long Span		Short Span		Long Span	
α_x		α_y		$M_x = \alpha_x w l_x^2$		$My = \alpha_y w l_y^2$	
Support	Span	Support	Span	Support	Span	Support	Span
0.0567	0.0426	0.047	0.035	9.08	6.83	10	7.4459

CHECK FOR DEPTH:- b = 1000 mm

Mumax = 0.133Fck*bd^2

= 0.133 x 25 x 1000 x 101^2

dreq = 54.850 mm < $d_{provided}$ d = 101 mm

% of Reinforcement

% of steel ,P_t = $f_ck/(2f_y)$ [1−√((1−4.598M

Short Span	
Support	Span
$\frac{25}{2x500}\left(1-\sqrt{1-\frac{4.598x9.08}{25x1000x101^2}}\right)$	$\frac{M25}{2x500}\left(1-\sqrt{1-\frac{4.598x6.83}{25x1000x101^2}}\right)$
= 0.2138	= 0.159

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PROJECT:	PROPOSED RESIDENTIAL BUILDING AT VGN LAKSHMI NAGAR, KOLAPAKKAM	DOCUMENT NO A23003-C-CB-01-DC-0001	REV 0
TITLE:	DESIGN OF SLAB	DESIGNED	CHECKED	APPROVED
CLIENT	M/S BUILD TECH ASSOCIATES	JSAN / DATE 02.08.2024	NKK / DATE 02.08.2024	NKK / DATE 02.08.2024

Long Span

Support	Span
$\frac{25}{2x500}\left(1-\sqrt{\frac{1-4.598x10}{25x1000x93^2}}\right)$	$\frac{M25}{2x500}\left(1-\sqrt{\frac{1-4.598x7.445946251 3625}{25x1000x93^2}}\right)$
= 0.2817	= 0.20645

Ast required

Short Span				Long Span			
Support		Span		Support		Span	
0.21/100x1000x101		0.159/100x1000x101		0.28/100x1000x93		0.206/100x1000x93	
=	215.91	=	160.59	=	261.9621913	=	191.994262

Design Required

Design Result along short Span					
At support	Provide	Y-	8	230	mm c/c
At span	Provide	Y-	8	310	mm c/c

Design Result along long Span					
At support	Provide	Y-	8	190	mm c/c
At span	Provide	Y-	8	260	mm c/c

Design Provided

Design Result along short Span					
At support	Provide	Y-	8	100	mm c/c
At span	Provide	Y-	8	100	mm c/c

Design Result along long Span					
At support	Provide	Y-	8	100	mm c/c
At span	Provide	Y-	8	100	mm c/c

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CHECK FOR DEFLECTION CONTROL:-

$A_{stprovided}$ = Spacing/3.14d^2

Short Span				Long Span			
Support		Span		Support		Span	
230/ 3.14 x 8 x 8		310/ 3.14 x 8 x 8		190/ 3.14 x 8 x 8		260/ 3.14 x 8 x 8	
=	502.65	=	502.66	=	502.65	=	502.655

Short Term Deflection

as per cl. C-2 p.no. 88 I_{eff} = $I_f/(1.2-M_r/M\ z/d\ (1-x/d)\ b_w/b)$

Reinforcement ratio	p	=	Ast/bd		
		=	502.66 / 1000 x 101	=	0.005
Modulus of elasticity	Ec	=	5000 √fck		
		=	5000 x √25	=	25000 KN/mm^2
Modulur ratio	m	=	Es/Ec		
		=	200000 / 25000	=	8
Neutral axis factor	k	=	$\sqrt{(2pm+(pm)^2)}$-pm		
		=	$\sqrt{(2x0.005x8) + (0.005x8)^2}$ - (0.005x8)	=	0.246
Lever arm factor	j	=	1-(k/3)		
		=	1-(0.246/3)	=	0.92
Lever arm	z	=	j d = 0.918x101	=	93
Neutral axis depth	x	=	k d = 0.246x101	=	24.9

PROJECT:	PROPOSED RESIDENTIAL BUILDING AT VGN LAKSHMI NAGAR, KOLAPAKKAM	DOCUMENT NO A23003-C-CB-01-DC-0001	REV	0
TITLE:	DESIGN OF SLAB	DESIGNED	CHECKED	APPROVED
CLIENT	M/S BUILD TECH ASSOCIATES	JSAN	NKK	NKK
		DATE 02.08.2024	DATE 02.08.2024	DATE 02.08.2024

Second moment of the area of the cracked section

I_{cr} = $(b\,(kd)^3/3) + (m\,Ast\,(d-kd)^2)$

$= \frac{1000 \times 0.246 \times 101\text{^}3}{3} + 8 \times 502.66 \times (101-0.246\times101)\text{^}2 = 28433814$ mm^4

Gross Moment of Inertia I_{gr} = $bD^3/12$

= $1000 \times 101\text{^}3/12 = 85858416.67$ mm^4

factor η = $z/d\,(1 - x/d)\,(bw/b) = 0.692$

Modulus of Rupture f_{cr} = $0.7\sqrt{fck}$ N/mm^2

= $0.7 \times SQRT25 = 3.5$

Distance from centroidal axis of gross section y_t = $D/2 = 62.5$ mm

Cracking Moment Mcr = $f_{cr}I_{gr}/y_t = 1$

= $\frac{3.5 \times 85858416.67}{62.5} = 4.81$ kNm/m

Effective second moment of area I_{eff} = $Icr/(1.2-(Mcr/M)\eta)$

= $\frac{28433813.79}{1/10\times0.692)} = 32790035$ mm^4

Icr < Ieff < Igr The section is safe | 28433813.79 < 32790035 < 85858416.67 |

Short term deflection due to total load

$\delta_{D+L} = \frac{1\ M\ L^2}{384\ Ec\ Ieff}$

$= \frac{1 \times 17.728443455625 \times 3.564\text{^}2}{48 \times 25000 \times 32790035} = 0.8$ mm

Short term deflection due to dead load alone

$\delta_D = \frac{5\ M_D\ L^2}{48\ Ec\ Ieff}$

$= \frac{1\times 6.25 \times 3.56\text{^}2}{384\times25000 \times 32790035} = 0.38$ mm

Short term deflection due to live load alone $\delta_L = \delta_{D+L} - \delta_D = 0.37$ mm

Deflection due to shrinkage: as per cl. C-3 p.no. 88

Ultimate shrinkage strain in cor Ecs — from cl. 6.2.4 p.n 16 = 0.0003

Shrinkage constant k_4 — from cl. C-3.1 p.n 88 = 0.051

Shrinkage constant k_3 = 0.125

Deflection due to shrinkage $\delta sh = k_3\,k_4\,\epsilon cs\,L^2/D$

$= \frac{0.125\text{x}0.05 \times 0.0003\text{x}(3.657\times1000)\text{^}2}{125.0} = 0.2$ mm

Deflection due to creep: as per cl. C-4 p.no. 88

Modulus of elasticity due to creep, Ece = $Ec/(1+\theta)$

= $25000/(1+1.6) = 9615.4$ N/mm^2

modulur ratio m = Es/Ece

= $200000/9615.38 = 20.8$

Neutral axis factor k = $\sqrt{(2pm+(pm)^2)}-pm$

= $\sqrt{2\text{x}0.005\text{x}20.8)+(0.005\text{x}20.8)\text{^}2}-(0.005\text{x}20.8) = 0.364$

Lever arm factor = $1-(k/3)$

j = $1-(0.364/3) = 0.879$

Lever arm = $j\,d = 88.78$ mm

Neutral axis depth z = $k\,d = 36.76$ mm

Second moment of the area of the cracked section x

I_{cr} = $(b\,(kd)^3/3) + (m\,Ast\,(d-kd)^2)$

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$= \frac{(1000x(0.364x101)^3}{3} + 20.8x10x(101-0.364x101)^2 = 59704746$ mm^4

$\eta_{factor} = z/d(1-x/d)(bw/b)$

$= (88.78/101) \times (1-36.76/101) = 0.559$

Effective second moment of area, $I_{eff\ D+L} = Icr/(1.2-(Mcr/M)\eta)$

$= \frac{59704746}{(1.2-(4.81/10X0.559)} = 49753961$ mm^4

Icr and Ieff are less than Igr, Hence it is ok

Initial plus creep deflection due to permenent loads

$a_{i,cc} = (1/384)(M_D L^2/(EceI_{effD+L}))$

$= \frac{1}{384} \times \frac{6.24660855375x10^6x3.657}{(3.564x1000)^2/(9615.38x49753961.34)} = 0.68$ mm

Short term deflection due to permanent loading

$a_i = (W_{DL}/W)\delta_{(D+L)}$

$= (3.605 / 15.9075) \times 0.37 = 0.3964$ mm

Deflection due to creep $= a_{cc} = a_{i,cc} - a_i = 0.29$ mm

$= L/250 = 14.63$ mm

Permissible final deflection due to all loads

Permissible deflection that occurs after construction

L/350 or 20mm which ever is less $= 10.45$ mm

Actual deflection due to all loads $= \delta_{D+L}+\delta_{sh}+a_{cc} = 1.24$ mm

Actual deflection is less than Permissible deflection hence safe

Actual deflection that occurs after constructi $= \delta_L+\delta_{sh}+a_{cc} = 0.86$ mm

Actual deflection is less than Permissible deflection hence safe

CHECK FOR CRACK WIDTH:-

$W_{cr} = \frac{3a_{cr} \times \varepsilon_m}{1 + \frac{2(a_{cr}-C_{min})}{h-a}}$

$a_{cr} = [[0.5S]^2 + C^2_{min}]^{1/2}$

Spacing of Ba $= 20$

$a_{cr} = 23$

$\varepsilon_m = \varepsilon_1 - \frac{b(h-a)(g-a)}{3E_s A_s (d-a)}$

$\varepsilon_1 = \frac{Fs}{Es} \times \frac{(h-a)}{(g-a)}$

$F_s = \frac{m \times Moment\ (d-a)}{I_r}$

$\frac{b\,a^2}{2} = m\ Ast\ (d-a)$

$a^2 = \frac{2m\ Ast\ d - 2m\ Ast\ a}{b}$

$= \frac{2 \times 20.8 \times 502.66 \times 101 - 220.8 \times 502.66a}{1000}$

$= \frac{418213.12 - 10898\ a}{1000}$

$a = 1$

$F_s = \frac{20.8 \times 10 \times (101-1)}{59704746} = 281.36$ N/mm^2

$\varepsilon_1 = \frac{281.36}{2.00E+05} \times \frac{(125-1)}{(50-1)} = 0.0036$

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PROJECT:	PROPOSED RESIDENTIAL BUILDING AT VGN LAKSHMI NAGAR, KOLAPAKKAM	DOCUMENT NO				REV	0
		A23003-C-CB-01-DC-0001					
TITLE:	DESIGN OF SLAB	DESIGNED		CHECKED		APPROVED	
CLIENT	M/S BUILD TECH ASSOCIATES	JSAN		NKK		NKK	
		DATE	02.08.2024	DATE	02.08.2024	DATE	02.08.2024

$$\varepsilon_m = 0.0036 - \frac{1000 \times (125-1)\times(50-1)}{3 \times 200000 \times 502.66 \times (101-1)} = 0.00339$$

$$W_{cr} = \frac{3 \times 23 \times 0.00339}{1 + \frac{2 \times (23-20)}{125-1}} = 0.1726$$

Limiting Crack width = 0.300

safe

CHECK FOR SHEAR:-

Average effective depth = 97 mm

$V_U = W_U(0.5 \times L_X - d_{avg})$

= 15.91 x (0.5 x 3.173 - (97 / 1000))

= 24 kN

$\tau_v = V_u/bd$ = 0.24

$(Pt)_{Provided}$ = 0.28

Sp-24-126 $\beta = 0.8f_{ck}/6.89p_t$ (but Sould not less than 1)

$\beta = \frac{0.8 \times 25}{6.89 \times 0.28}$

β = 10

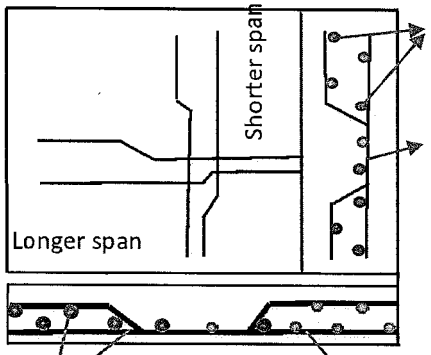
τ_c = 0.85*sqrt(0.8*fck)*(sqrt(1+5β)-1)/6β

= 0.85 x SQRT(0.8x25)x (SQRT(1+5x)-1)/6x10.37

= 1.21 Safe

REINFORCEMENT DETAILS:-

Two way slab



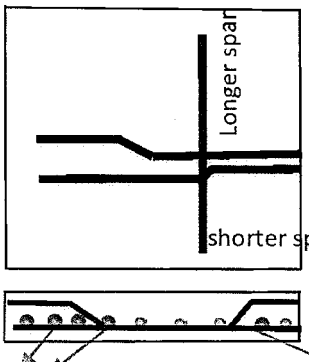
Provide 8 mm ϕ 230 mm c/c

Provide 8 mm ϕ 310 mm c/c

Provide 8 mm ϕ 190 mm c/c

Provide 8 mm ϕ 260 mm c/c

One way slab



Provide 8 mm ϕ 230 mm c/c

Provide 8 mm ϕ 190 mm c/c

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PROJECT:	PROPOSED RESIDENTIAL BUILDING AT VGN LAKSHMI NAGAR, KOLAPAKKAM	DOCUMENT NO				REVISION	0
		A23015-C-RB-01-DC-0002					
TITLE:	DESIGN OF STAIRCASE	DESIGNED		CHECKED		APPROVED	
CLIENT	M/S BUILD TECH ASSOCIATES	JSAN		NKK		NKK	
		DATE	02.08.2024	DATE	02.08.2024	DATE	02.08.2024

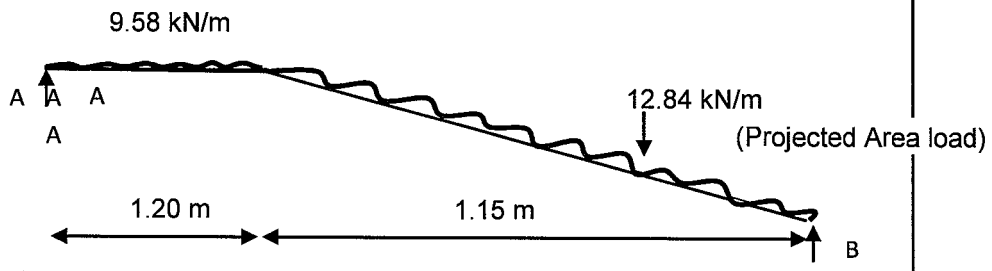
Input:

Unit Weight of concrete(RCC)	=	25	KN/m^3
Unit Weight of concrete(PCC)	=	24	KN/m^3
Unit Weight of Masonary work	=	20	KN/m^3
Tensile strength of steel	=	500	N/mm^2
Compressive strength of concrete	=	20	N/mm^2
Main bar Dia	=	12	mm
Distribution bar Dia	=	8	mm

Stair Case Details:

Tread		=	0.30	m
Riser		=	0.18	m
Thickness of slab		=	0.175	m
Efeective depth of slab	= 0.175 - 0.025 - 0.006	=	0.144	m
Thickness of finishing		=	0.05	
Clear cover		=	25	mm
X_{umax}/d		=	0.460	

A **Flight 1 Design:**



Loads o landing:

i)Dead load:

Self weight		=	4.38	kN/m^2
Finishing load		=	1.20	kN/m^2
Total load per m run	4.375 + 1.2	=	5.58	kN/m^2

ii)Live load:

Railway building class		=	4	kN/m^2

IS 875 Part 2 Table 2 (Corridors, passages, lobbies, staircases)

UnFactored Loads on Landing slab		=	**9.58**	kN/m^2

Loads on Going (or) Horizontal portion on Projected Plan Area

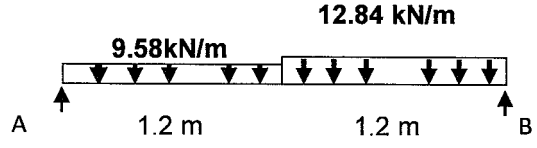
i)Dead load:

Self weight		=	5.065	kN/m^2
Finishing load		=	1.583	kN/m^2
Load due to Steps		=	2.188	kN/m^2
Total load per m run		=	**8.836**	kN/m^2

ii)Live load:

Railway building class		=	4	kN/m^2
Unactored Loads on Going on Projected Plan Area		=	**12.84**	kN/m^2

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PROJECT:	PROPOSED RESIDENTIAL BUILDING AT VGN LAKSHMI NAGAR, KOLAPAKKAM	DOCUMENT NO				REVISION	0
		A23015-C-RB-01-DC-0002					
TITLE:	DESIGN OF STAIRCASE	DESIGNED		CHECKED		APPROVED	
CLIENT	M/S BUILD TECH ASSOCIATES	JSAN		NKK		NKK	
		DATE	02.08.2024	DATE	02.08.2024	DATE	02.08.2024

As per Clause 33.1 (b) of IS : 456 -2000 , the effective span of stair is taken as = 2.35 m

Reaction at Support B = $\frac{9.575\times1.2^2/2+12.84\times1.15\times(0.575+1.2)}{2.35}$ = 14.08 kN

Reaction at Support B = 12.17 kN

Shear Force is zero at = 14.08-12.84X = 0

X = 1.10 From B

Maximum B.M = (14.08x1.1) -12.84x(1.1)^2/2 = 7.73 kN-m

Design Bending Moment = 40.55 kN-m

Check For Depth

d_{lim} = 123.2 mm

$d_{provided}$ = 144.0 mm

= **SAFE**

Reinforcement Details

$R=M_u/bd^2$ = 1.956

$(Pt)_{Required}$ = **0.516** %

Area of steel required (**$A_{st)\ Required}$**= = 743 mm^2

a_{st} = 113 mm^2

Spacing req = 152.214 mm

$Spacing_{pro}$ = 150 mm

Provide12mm bar at 150mm C/C

$(Pt)_{Provided}$ = = **0.524**

SAFE

Min steel required = **210** mm^2

Provide8mm bar at 225mm C/C

Check For Shear

24-126

$(Pt)_{Provided}$ = = 0.524

$\beta=0.8f_{ck}/6.89p_t$(but Sould not less than 1)

β = 4.435

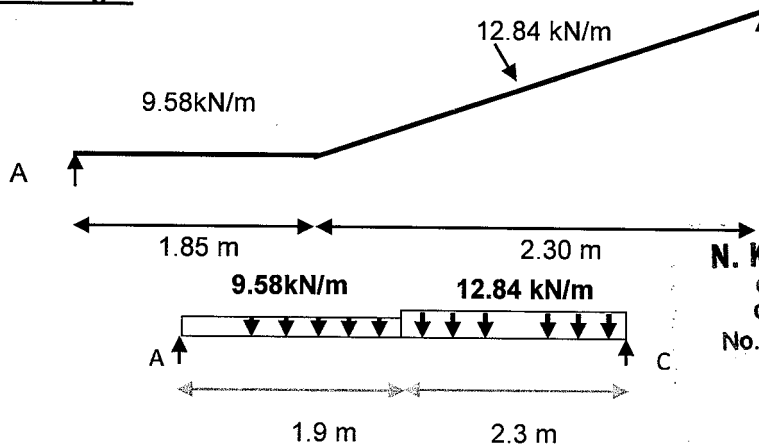
Tc=0.85*sqrt(0.8*fck)*(sqrt(1+5β)-1)/6β Tc = 0.487 Nmm^2

Design Shear Force (V_u) = **18.35** kN

T_v = 0.127 Nmm^2

Tv < Tc **SAFE**

Design of 2nd flight



As per Clause 33.1 (b) of IS : 456 -2000 , the effective span of stair is taken as

= 1.85+2.3 = 4.15 m

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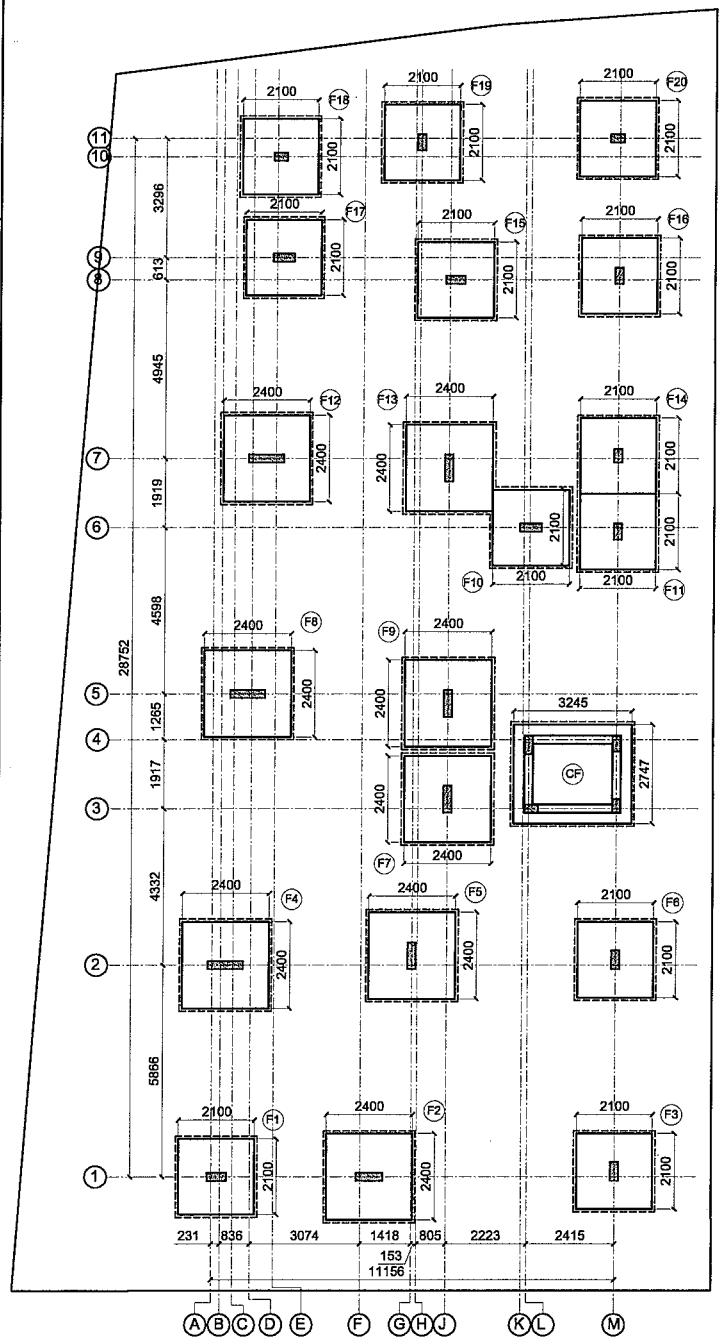
PROJECT:	PROPOSED RESIDENTIAL BUILDING AT VGN LAKSHMI NAGAR, KOLAPAKKAM	DOCUMENT NO				REVISION	0
		A23015-C-RB-01-DC-0002					
TITLE:	DESIGN OF STAIRCASE	DESIGNED		CHECKED		APPROVED	
CLIENT	M/S BUILD TECH ASSOCIATES	JSAN		NKK		NKK	
		DATE	02.08.2024	DATE	02.08.2024	DATE	02.08.2024

	Reaction at Support C = $\frac{(9.575\times1.85^2/2)+12.84\times2.3\times(1.15+1.85)}{4.15}$		=	**25.29**	kN
	Reaction at Support A =		=	**21.95**	kN
	Shear Force is zero at = 25.29-12.84X		=	0	
		X	=	1.97 From B	
	Maximum B.M = (25.29x1.97) -12.84x(1.97)^2/2		=	24.91	kN-m
	Design Bending Moment		=	**41.00**	
	Check For Depth				
	d lim=		=	123.87	mm
	d provided		=	144.00	mm
				SAFE	
	Reinforcement Details				
	$R=M_u/bd^2$		=	1.977	
	$(Pt)_{Required}$		=	**0.523**	%
	Area of steel required (**$A_{st)\ Required}$**=		=	753	mm^2
	a_{st}		=	113	mm^2
	Spacing req		=	150	mm
	$Spacing_{pro}$		=	150	mm
	Provide12mm bar at 150mm C/C				
	$(Pt)_{Provided}$ =		=	**0.524**	
				SAFE	
	Distribution steel				
	Min steel required		=	**210**	mm^2
	Provide8mm bar at 225mm C/C				
	Check For Shear				
24-126	**$(Pt)_{Provided}$ =**		=	0.524	
	$\beta=0.8f_{ck}/6.89p_t$(but Sould not less than 1)				
		β	=	1.000	
	Tc=0.85*sqrt(0.8*fck)*(sqrt(1+5β)-1)/6β	Tc	=	0.821	Nmm^2
	Design Shear Force (V_u)		=	**35.16**	kN
	T_v		=	0.244	Nmm^2
	Tv < Tc			**SAFE**	

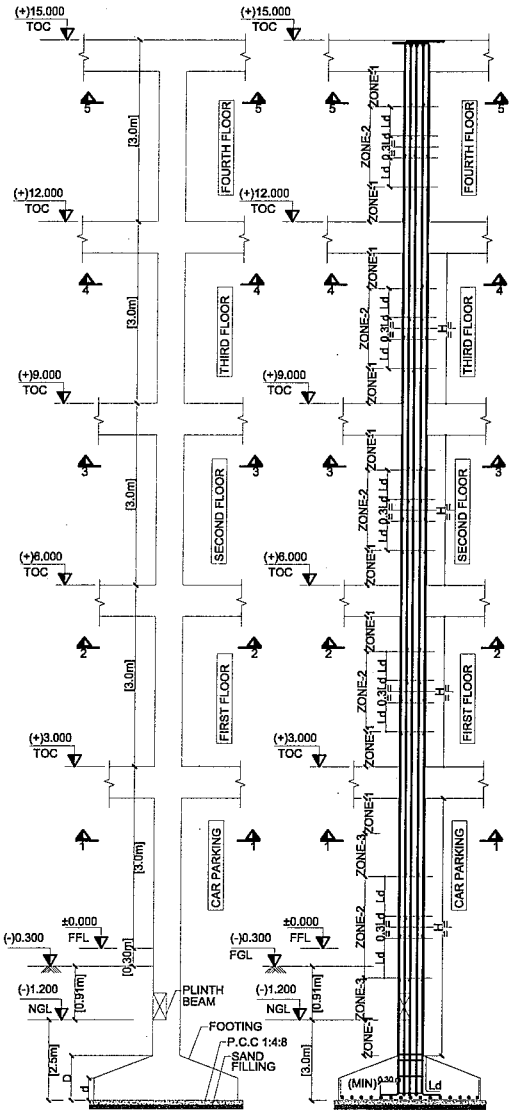
N. KARIKALAN, M.E.,(Struc.Engg.), MIE
CMDA Reg. No : SE/GR-I/19/03/012
Chartered Engineer (M-1644542/19)
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LAYOUT & NUM. DET. OF FOOTING



L.S OF COLUMN

L.S OF COLUMN (TYP.RC DETAILS)

ZONE-1, TIES SPACING IS CONSIDERED AS 100MM C/C
ZONE-2, TIES SPACING IS CONSIDERED AS 150MM C/C
ZONE-3, TIES SPACING IS CONSIDERED AS 200MM C/C

Ld - DEVELOPMENT LENGTH

GRADE \ Ø(mm)	8	10	12	16	20
M20	1'4.5"	1'9"	2'0"	2'7.5"	3'4.5"
M25	1'3"	1'6"	1'9"	2'4.5"	2'10.5"

REINFORCED CEMENT CONCRETE(RCC):-

GRADE OF CONCRETE	MINIMUM CEMENT CONTENT (kg/m3)	WATER CEMENT RATIO
M20	300	0.55
M25	300	0.50
M30	320	0.45

PLAIN CEMENT CONCRETE(PCC):-

	MINIMUM CEMENT CONTENT (kg/m3)	WATER CEMENT RATIO
PCC	240	0.60

THE MINIMUM NET S.B.C AT 2.5m BELOW NGL IS 24 T/SQ.M AS PER SOIL REPORT DONE BY CHENNAI SOIL LAB

BUILDING CO-ORDINATES TO BE DECIDED BY ARCHITECT / CLIENT

IN CASE OF ANY OTHER SERVICE LINE FOULING WITH FOUNDATION, THEN SAME TO BE REPORTED TO STRUCTURAL DESIGNER IMMEDIATELY.

THE STRUCTURE IS DESIGNED FOR STILT +4 FLOORS ONLY

NOTES:-

1. ALL DIMENSIONS ARE IN FEET AND INCHES & LEVELS ARE IN METRES.
2. "T" DENOTES HYSD BARS (Fe 500 / Fe 500 D) CONFORMING TO IS :- 1786: 2008
3. UOS MINIMUM CLEAR COVER TO ANY REINFT. SHALL BE AS FOLLOWS :

MEMBER MKD.	CLEAR COVER
FOOTING	50 mm
COLUMN	40 mm
BEAMS	30 mm OR BAR DIA WHICHEVER IS GREATER
SLAB	25 mm OR BAR DIA WHICHEVER IS GREATER

4. GRADE OF CONCRETE FOR FOOTIING,COLUMN,BEAM & SLAB SHALL BE M20 (MIN.). GRADE OF CONCRETE FOR LINTEL & SUNSHADE SHALL BE M20 (MIN.).
5. LAPS SHALL BE STAGGERED SUCH THAT NOT MORE THEN 50% OF REINFORCEMENT IS LAPPED AT ANY LOCATION.
6. TOP BARS IN BEAMS & SLABS SHALL BE LAPPED AT MID SPAN AND NOT AT SUPPORT ZONE.
7. BOTTOM BARS IN BEAMS & SLABS SHALL BE LAPPED AT END ONE THIRD SPAN.
8. COL. REINFT. LAPS SHALL BE STAGGERED & DONE JUST ABOVE BEAM COL. INTERSECTIONS UOS.
9. ALL HOOKS, BENDS AND LAPS SHALL BE AS PER IS :- 2502:1963 & IS13920-1993 UNLESS OTHERWISE INDICATED.

REV.NO.	DESCRIPTION	DESIGNED	DRAWN	CHECKED	APPROVED
A	ISSUED FOR APPROVAL	JSAN 02.08.24	JSAN 02.08.24	NKK 02.08.24	NKK 02.08.24

SIVASANKARI STRUCTURAL CONSULTANT
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E-MAIL ID: sivasankari.structural@gmail.com

CLIENT : M/S BUILD TECH ASSOCIATES	CONSULTANT : SIVASANKARI STRUCTURAL CONSULTANT
PROJECT : (S+4)PROPOSED RESIDENTIAL BUILDING AT KOLAPAKKAM	CONCEPT ARCHITECT : SNS ASSOCIATES
SUPPLIER/ CONTRACTOR : M/S BUILD TECH ASSOCIATES	

JOB No : A23015

	NAME	SIGN	DATE
DSGN	JSAN		02.08.24
DRWN	JSAN		02.08.24
CHKD	NKK		02.08.24
APPD	NKK		02.08.24

TITLE:
RESIDENTIAL BUILDING
LAYOUT &NUM.RC DET. OF FOOTING COLUMN

SECTION / ELEVATION MARKING
DRAWING NUMBER
SCALE 1:100

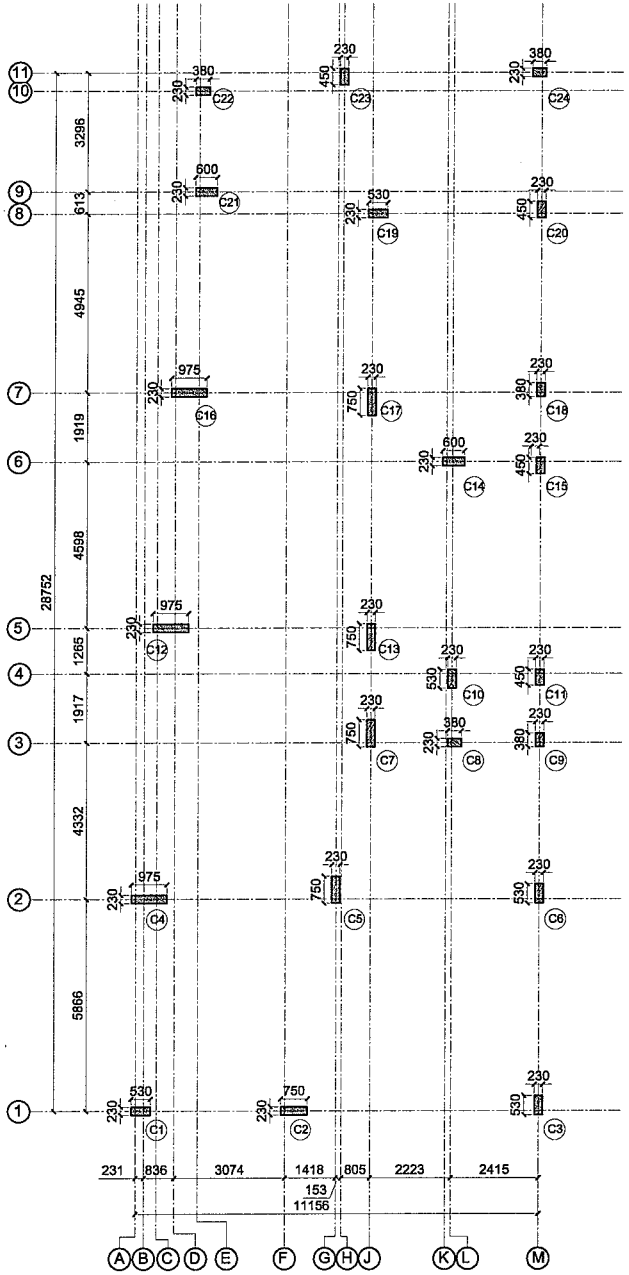
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PROJECTION | SIZE A3 | REV A

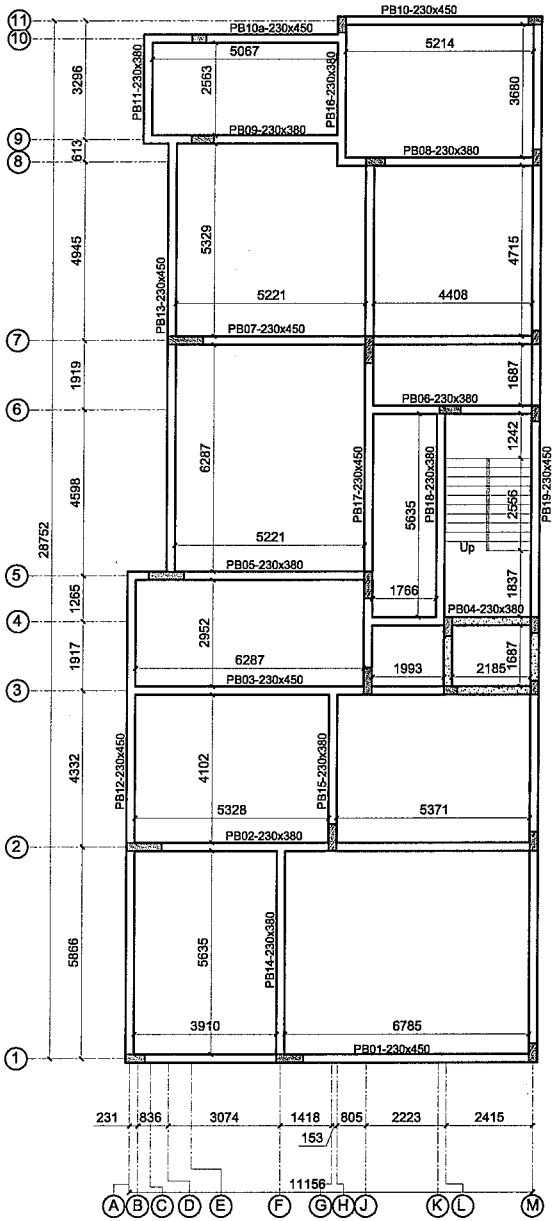
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SIVASANKARI STRUCTURAL CONSULTANT

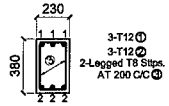
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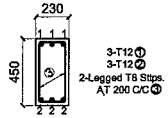
LAYOUT & NUM. DET. OF COLUMN



LAYOUT & NUM. DET. OF PLINTH BEAM



PLINTH BEAM -230x380



PLINTH BEAM -230x450

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APPD	NKK		02.08.24

TITLE:
RESIDENTIAL BUILDING
LAYOUT & NUM.R.C DET. OF COLUMN & PLINTH BEAM

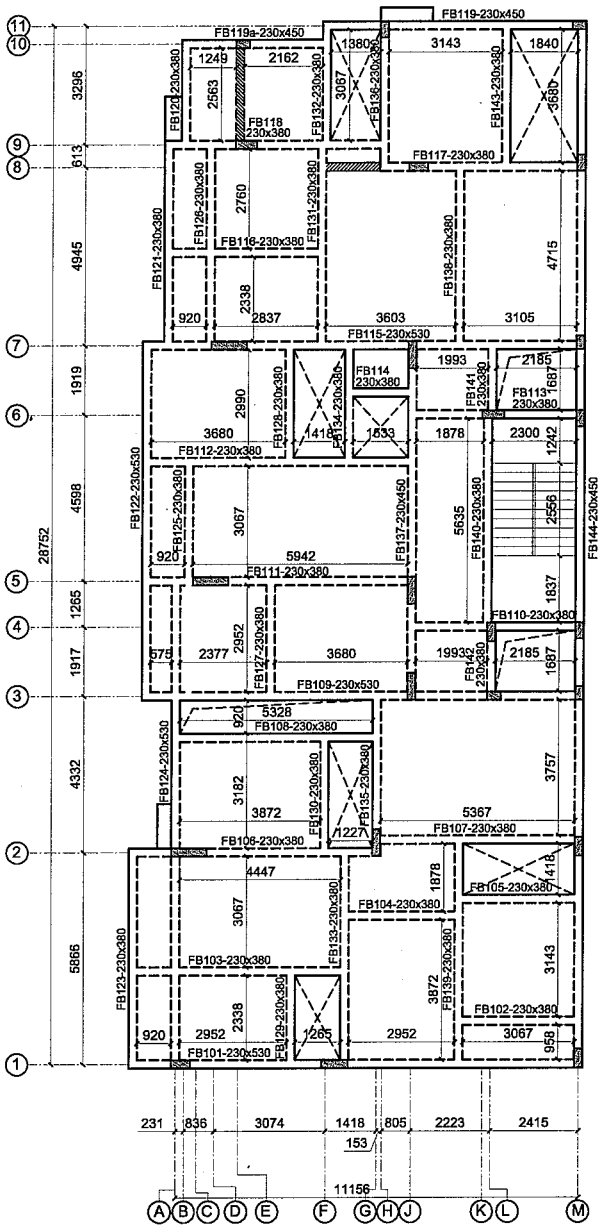
SECTION/ ELEVATION MARKING
DRAWING NUMBER
SCALE 1:100

DRAWING No.	PROJECTION	SIZE	REV
A23015 - C -RB - 04 - 0002		A3	A

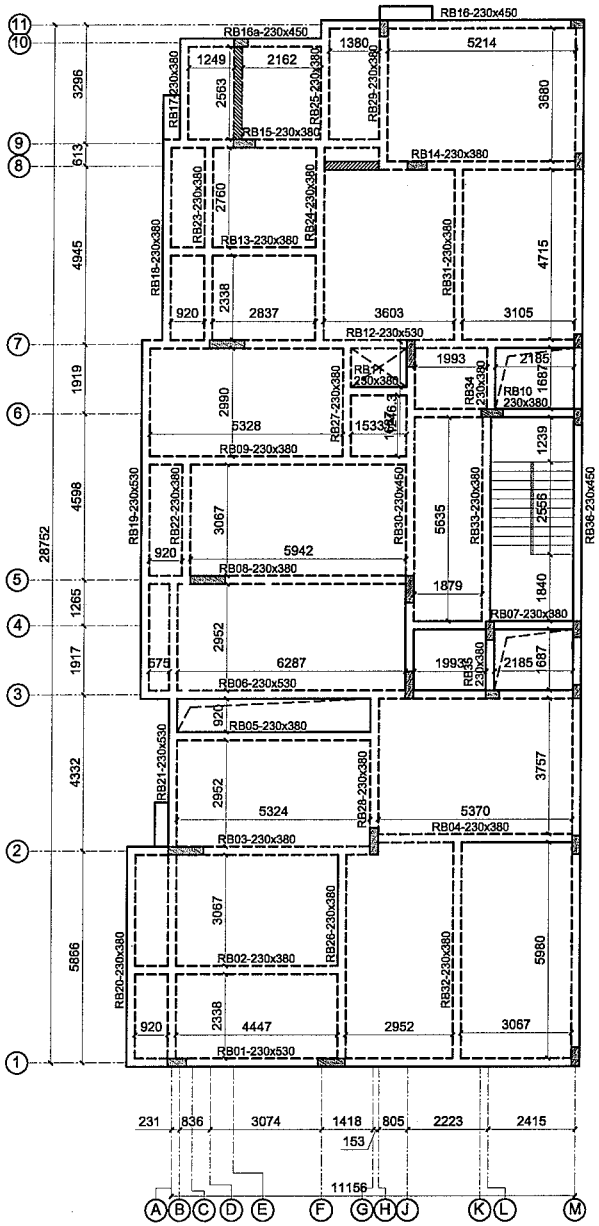
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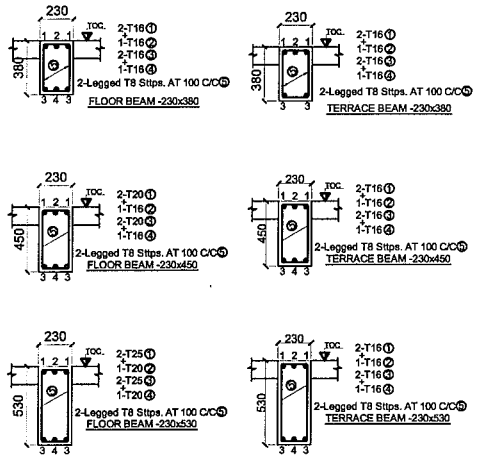
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LAYOUT & NUM. DET. OF FLOOR BEAM
(SLAB THICKNESS 125mm)



LAYOUT & NUM. DET. OF TERRACE BEAM
(SLAB THICKNESS 125mm)



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TITLE:
RESIDENTIAL BUILDING
LAYOUT,NUM & RC DET. OF FLOOR BEAM & TERRACE BEAM

SECTION / ELEVATION MARKING
DRAWING NUMBER
SCALE 1:100

DRAWING No. A23015 - C -RB - 04 - 0003
PROJECTION
SIZE A3
REV A

RELEASED FOR ☐PRELIMINARY ☐ TENDER ☐INFORMATION ☑APPROVAL ☐CONSTRUCTION



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Compliance & Special Services

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Ownership, Compliance & Legal Verification

Encumbrance Certificate (EC) Search, Find Your SRO, Temple Property Check, WAQF Property Check, View Patta/Chitta, View FMB, Patta Analyzer, EC Analyzer

Zoning & Land Use Analysis

Land Use Zone Finder, CRZ Zone Check, Aquifer Zone Check, CBA Check, EWS Check, Road Width Check, Proposed Road Widening Check, FSI Calculator

Property Documentation & Workflow Tools

Forms & Templates, Document Generator, Document Translation, Dictionary

Vastu & Property Management Tools

Home Vastu Shastra, Plot Vastu Shastra, Property Management

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