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Dwaraka Darshan

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04.09.2024

To
The Member Secretary,
CMDA,
Chennai – 600 008.

CERTIFICATE

The proposed Residential building with **Stilt+5 Floors Residential building with 10 Dwelling units, at Block No.:79, Dwaraka flats, Ambattur Estate-Thirumangalam Road, Anna Nagar Western Extension, Chennai 600101, Comprised in old S.No:289 Part of Padi Village, S.Nos:297/3B, 297/1A Part, 1B Part, 2A Part, 2B Part, 290 Part, T.S.No: 87/9, 87/10, 87/11, 87/12, Block No: 9, Ward-H of Mogappair village, Ambattur taluk, Thiruvallur District within the limits of Greater Chennai Corporation,** has been designed as per IS Codal regulations and the construction will be carried out as per Standard Engineering Practice and hence the building is certified to be structurally safe & sound.

The entire structure shall be designed with all safety precautions as per IS Codal regulation to support the slab and walls at every floor level.

V.R. RAVINDRAN.
Structural Consultant
V R P CONSULTANTS

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Design of Structure:

The structure has been modeled in software "ETABS". The columns and beams have been modeled as per plan. The slab has been given as plate model and loadings are applied as per IS Codal regulations. The wall load has been applied on beams as per plan.

The structure has been analysed as three-dimensional structure. The earthquake forces are applied as per IS 1893-2002. The ductile detailing has been followed as per IS 13920-1993.

The loads on columns are taken and foundation is designed as per IS Code. Beams and columns are designed by ETABS. The floor slab is designed manually using IS Codal regulations.

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STABILITY CERTIFICATE

It is certified that Residential building **Stilt+5 Floors Residential building with 10 Dwelling units, at Block:79, Dwaraka flats, Ambattur Estate-Thirumangalam Road, Anna Nagar Western Extension, Chennai 600101, Comprised in old S.No:289 Part of Padi Village, S.Nos:297/3B, 297/1A Part, 1B Part, 2A Part,2B Part, 290 Part, T.S.No: 87/9, 87/10, 87/11, 87/12, Block No:9, Ward-H of Mogappair village, Ambattur taluk, Thiruvallur District within the limits of Greater Chennai Corporation,** has been designed to resist earthquake, check has been made and found safe and I hereby certify that

1. The least dimension of the column is 300 mm.
2. The grade of concrete is **M-25**.
3. The design and analysis is done using the codes of practice for loading standards as per IS: 875 Parts and Seismic force as per the code IS 1893-2002 & IS 456:2000

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DESIGN OF SLAB

Data:

Concrete	M25		fck	=	25	N/mm²
Steel	Fe 500		fy	=	500	N/mm²
Shorter Span			Lx	=	3.72	m
Longer Span			Ly	=	3.84	m
Assuming Depth of Slab			D	=	150	mm
Assuming Dia of reinf.				=	8	mm
Clear Cover				=	20	mm
Eff. Depth of Slab			d	=	126	mm
Eff.Shorter Span			l_x	=	3.77	m
Eff. Longer Span			l_y	=	4.38	m

1. Load Calculations:

Self weight of slab	0.15	x	25.00	=	3.75	kN/m²
Floor Finish load	0.075	x	20.00	=	1.50	kN/m²
Live load				=	2.00	kN/m²
Total load				=	7.30	kN/m²
Load Factor				=	1.50	
Factored Load	2			=	10.95	kN/m²
Span Ratio			ℓ_y/ℓ_x	=	4.38 / 3.77	
				=	1.16	

2. Bending Moment Calculation:

Support Condition	**2**	One Short Edge Discontinuous			
From IS 456-2000 Table 26	2	α_{Nx}	=	0.046	
		α_{Px}	=	0.034	
		α_{Ny}	=	0.037	
		α_{Py}	=	0.028	

Shorter Span:

Negative Moment (Support)	M_{Nx}	=	$\alpha_{Nx} \times w_u \times l_x^2$	
		=	$0.046 \times 10.95 \times 3.77^2$	
		=	7.17	kN.m
Positive Moment (Mid span)	M_{Px}	=	$\alpha_{Px} \times w_u \times l_x^2$	
		=	$0.034 \times 10.95 \times 3.77^2$	
		=	5.36	kN.m

Longer Span :

Negative Moment (Support)	M_{Ny}	=	$\alpha_{Ny} \times w_u \times l_x^2$	
		=	$0.037 \times 10.95 \times 3.77^2$	
		=	5.76	kN.m

Positive Moment (Mid span)	M_{Py}	=	$\alpha_{Py} \times w_u \times l_x^2$
		=	0.028 x 10.95 x 3.77²
		=	4.36 kN.m

Reinforcement:

Required in Shorter Span	Reinf Required		Reinf Provided
Support	8Ø @ 330mm c/c	-	8 dia @ 150 mm c/c
Mid span	8Ø @ 330mm c/c	-	8 dia @ 150 mm c/c
Required in Longer Span			
Support	8Ø @ 330mm c/c	-	8 dia @ 150 mm c/c
Mid span	8Ø @ 330mm c/c	-	8 dia @ 150 mm c/c

3. Check for Shear:

As per BS:8110 Shear coefficients

Max Shear coeff. for continuous edge in short direction	γcx =	0.41
Shear coefficient for discontinuous edge in short direction	γdcx =	0.00
Shear coefficient for continuous edge in long direction	γcy =	0.36
Shear coefficient for discontinuous edge in long direction	γdcx =	0.24

S.F for continuous edge in short direction	Vcx	=	$\gamma cx * \ell x * W$
Maximum Shear Force	Vu	=	0.41 x 3.770 x 10.95
		=	16.87 kN
S.F for discontinuous edge in short direction	Vdcx	=	$\gamma dcx * \ell x * W$
		=	0.00 x 3.770 x 10.95
		=	0.000kN
S.F for continuous edge in long direction	Vcy	=	$\gamma cy * \ell x * W$
		=	0.36 x 3.770 x 10.95
		=	14.861kN
S.F for discontinuous edge in long direction	Vdcy	=	$\gamma dcy * \ell x * W$
		=	0.24 x 3.770 x 10.95
		=	9.908kN

S.F for continuous edge in short direction	Vu	=	16.865	
Design Shear stress τv	Vu/bd	=	16865 / (1000 x 126)	
		=	0.134	N/mm²

Reinforcement provided in the slab is		=	8Ø @ 150 mm c/c
% of reinforcement provided	pt	=	0.266
IS:456 Table 19			
Allowable shear stress	τc	=	0.37 N/mm²
As per IS:456 cl. 40.2.1.1 k value for slab		=	1.30
Design shear stress for concrete	k x τc	=	1.30 x 0.375
		=	0.487 N/mm²

Actual Shear Stress < Design Shear Stress 0.134 < 0.487

SAFE

4. Check for deflection

Allowable span / depth Ratio		l/d	=	26	
% of Tension reinforcement		pt	=	0.266	
Corresponding Compression reinf.		Bar dia	=	0	mm
		spacing	=	150	mm
% of Compression reinforcement		pc	=	0.000	
Modification factor for Tension reinf.	F1	=	2.00	(From Fig 4 in IS 456 - 2000)	
Modification factor for Comp. reinf.	F2	=	1.00	(From Fig 5 in IS 456 - 2000)	
Modified span/depth ratio		=	l/d x F1 x F2		
		=	26 x 2.00 x 1.00		
		=	**52.00**		
Actual span/depth ratio		=	3770 / 126		
		=	**29.92**		

Actual span/depth ratio < Modified span/depth ratio 29.92 < 52.00

SAFE

DESIGN OF BEAM

Data:

Effective span		=	3.95 m
		=	3950 mm
Breadth of beam		=	230 mm
Overall depth (assumed)		=	450 mm
Characterstic strength of steel			
	fy	=	500 N/mm²
	fck	=	25 N/mm²
Cover		=	25 mm

Total load

Dead load from slab		=	3.75 kN/m²
Floor finish		=	1 kN/m²
Partition		=	1 kN/m²
Total dead load		=	5.75 kN/m²
Live load		=	2 kN/m²
	Total load	=	7.75 kN/m²

Effective depth = **d** = $450 - 25 - \frac{25}{2}$

= 412.5 mm

Distance of centroid of Compression reinforcement from the extreme compression fibre =

d' = 25 + (25/2)

= 37.5 mm

$$\frac{d'}{d} = \frac{37.5}{412.5} = 0.1$$

Minimum Reinforcement

The min. area of tension reinforcement shall not be less than the given,

$$\frac{As}{bd} = \frac{0.85}{fy}$$

Where ,

As	=	Min. area of tension reinforcement
b	=	Breadth of the beam or breadth of web of T-beam
d	=	Effective depth

fy = Characterstic strength of reinforcement

$$As = \frac{0.85 \times b \times d}{fy}$$

$$= \frac{0.85 \times 230 \times 412.5}{500}$$

$$= 161.29 \text{ mm}^2$$

Refering Design Aids **IS:456**, The reinforcement as shown in the sketch

Overall depth of beam = $\frac{1}{12}$ to $\frac{1}{20}$ of the span

Provide over all depth = 450 mm

Width of rib = $\frac{2}{3}$ (depth of rib) = $\frac{2}{3}$ (450 - 150)

= 200 mm

Loading on the beam

From the slab = $\frac{7.75 \times 3.950}{3}$

= 10 kN/m

D.L of the rib = 0.23 x 0.3 x 25 kN/m

= 1.8 kN/m

Total = 11.8 kN/m

Provide 20 mm dia bars at clear cover 25 mm

Effective cover = 25 + 10

= 35 mm

Effective depth d = 415 mm

Center to Center bearing = 3.95 m

Effective span = 3.95 m

Max. BM = $\frac{11.8 \times 3.950 \times 3.950}{8}$

= 23 kN.m

Req.depth **d** = $\left\{ \frac{1.5 \times 23 \times 10 \wedge 6}{2.76 \times 230} \right\}^{½}$

= 233 mm

Provided depth, D = 450 mm

$$\frac{Mu}{bd^2} = \left\{ \frac{1.5 \times 23 \times 10 \text{ ^ } 6}{230 \times 415 \times 415} \right\}^{1/2}$$

$$= 0.9$$

$$pt = \frac{fck}{2 \times fy} \left\{ 1 - \left(\text{SQRT} \left[1 - \frac{4 \times (Mu/b \times d^2)}{0.87 \times fck} \right] \right) \right\} \times 100$$

$$= 0.22 \%$$

$$Ast = \left\{ \frac{0.22 \times 230 \times 415}{100} \right\}$$

$$= 210 \text{ mm}^2$$

Provided 3 - Y16

Check for Shear

$$V = \frac{1.5 \times 11.8 \times 3.950}{2}$$

$$= 35 \text{ kN}$$

$$\tau_c = 0.54 \text{ N/mm}^2$$

$$Vuc = \frac{0.54 \times 230 \times 412.5}{1000}$$

$$= 51.2 \text{ kN}$$

Vuc > Shear at support

Provide Y8 @ 150 c / c (stirrups)

DESIGN OF COLUMN

Data :

Total No. of floors		=	5
Total No. of Roof		=	6
Storey height		=	3.05 m
Breadth of column	b	=	300 mm
		=	0.3 m
Depth of column	D	=	530 mm
			0.53 m
	fck	=	25 N/mm²

Total axial load

Dead load & live load from one floor		=	140 kN
Total axial load all floors		=	6 x 140
		=	840 kN
Column dimension		=	300 x 530 mm
Self weight of column		=	0.3 x 0.53 x 6 x 25 x 3.1
		=	73 kN
Total load		=	(840 + 73) x 1.5
	Pu	=	1369.5 kN

Area of Reinforcement

	$\frac{d'}{D}$	=	0.1
	d'	=	53 mm
	$\frac{Pu}{fck.b.D}$	=	$\frac{1,370 \times 10^{3}}{25 \times 300 \times 530}$
		=	0.34
	$\frac{Mu}{fck.b.D^2}$	=	$\frac{1.7 \times 1.5 \times 15.4 \times 10^{6}}{25 \times 300 \times 530 \times 530}$
		=	0.02

Refering to chart in SP:16 for values

$\frac{Pu}{fck.b.D}$ = 0.34 ; $\frac{Mu}{fck.b.D^2}$ = 0.02

The value of $\frac{P}{fck}$ = 0.08

$$p = 0.08 \times 25$$
$$= 2$$

$$As = \frac{p \times b \times D}{100}$$

$$= \frac{2.00 \times 300 \times 530}{100}$$

$$= 3180 \text{ mm}^2$$

Provide 8 - Y25 bars

Design of Lateral Ties

Dia of lateral ties $= \frac{1}{4} \times 25 = 6$ mm

Hence 8 dia bar is adopted

Pitch is min. of the following - (as per IS:456)

a. Least lateral dimension = 300 mm
b. 16 x dia of main reinforcement = 16 x 25 = 400mm
c. 300mm

Hence provide Y8 @ 200 c/c
Similarly other columns are calculated.

DESIGN OF FOUNDATION

Design of Isolated footing for intermediate column

Data :

Column size	L	=	300	mm
	B	=	530	mm
Size of footing	L	=	2	m
	B	=	2	m
Safe Bearing capacity of soil	SBC	=	150	kN/m²
Depth of footing below GL		=	2.13	m
Density of soil	**g**	=	18	kN/m³
Characterstic strength of concrete	fck	=	25	N/mm²
Characterstic strength of Steel	fy	=	500	N/mm²
Depth of footing provided	D	=	0.68	m
cover		=	50	mm

1. Loading on footing

Axial load on column		=	913 kN
Self weight of footing (10%)		=	91.3 kN
Total load on footing	W	=	1004.3 kN
Adpot size of footing		=	2.00 x 2.00 m
		=	4 m²
Soil pressure,	$\frac{P}{A}$	=	$\frac{1004.3}{4}$
		=	142 kN/m²
Gross safe bearing capacity		=	187.5 kN/m²

Hence safe

2. Moment calculation

Face of the column		=	$\frac{2.00 - 0.30}{2}$
		=	0.85 m
Design pressure		=	87 kN/m²
Moment	M	=	$\left[\frac{87 \times (0.85)^2}{2}\right]$
		=	31 kN.m
Factored moment	Mu	=	46.5 kN.m

3. Area of Reinforcement

Effective depth d = 620 mm (or) 0.62 m

Dia of bar provided Ø = 20 mm

Provided spacing = 100 mm

$$\frac{Mu}{b.d^2} = \frac{47 \times 10\ ^{\wedge}6}{1000 \times 620 \times 620}$$

$$= 0.12$$

Percentage of tension reinforcement

$$pt = \frac{fck}{2 \times fy}\left[1 - \left(SQRT\left[1 - \frac{4 \times (Mu/b \times d^2)}{0.87 \times fck}\right]\right)\right] \times 100$$

$$= 0.03\ \%$$

Required, Ast $= \frac{0.03 \times 1000 \times 620}{100}$

$= 186\ mm^2$

Provided, Ast $= 3142\ mm^2$

$= 0.51\ \%$

Hence safe

4. Check for One way shear

Vu $= 1.5 \times 87.00 \times (((2.00 \times 0.5) - (0.30 \times 0.5)) - 0.62)$

$= 30$ kN

Permissible shear stress, $= \frac{30 \times 1000}{1000 \times 620}$

$= 0.05\ N/mm^2$

Allowable shear stress, $= 0.49\ N/mm^2$

Hence safe

5. Check for Punching shear

Punching force Pf $= \left[87 \times (4.0 - ((0.30 + 0.62) \times (0.53 + 0.62))\right] \times 1.5$

$= 384$ kN

Punching shear stress $= \frac{384 \times 1000}{620 \times 2 \times (300 + 530 + (2 \times 620))}$

$= 0.15\ N/mm^2$

Allowable shear stress $= 1.25\ N/mm^2$

Hence safe

EARTH QUAKE ANALYSIS

1. Calculation of horizontal seismic Coefficient

As per IS 1893 - 2002

$$Ah = \frac{z \cdot I}{2 \cdot R} \frac{Sa}{g}$$

Zone factor,	Z	=	0.16	(As per table 7)
Importance factor,	I	=	1	
Response reduction factor,	R	=	5	

$$\frac{sa}{g} = 2.5$$

Natural period, Ta = 0.075 h ¾

(Clause.7.6) = 0.075 x 18.3 ^0.75

= 0.66 sec

h = height of the building

$$Ah = \frac{0.16 \times 1 \times 2.5}{2 \times 5}$$

= 0.04

2. Calculation of Base shear & seismic forces

Base shear V = An ∑ Wi

= 0.040 x 913

= 36.5 kN

Total no of columns = 20

Shear per column = $\frac{36.5}{20}$

= 1.83

Seismic moment per column = $\frac{1.83 \times 3}{4}$

= 1.4 T-m

Column Provided = 300 x 530 mm with 8 - Y25

Hence safe

3. Checking for footing

SBC	=	15 T/m²
Soil pressure	=	14.2 T/m²

$$\frac{p}{A} \pm \frac{1.40 \times 6}{12}$$

= 14.2 ± 0.7

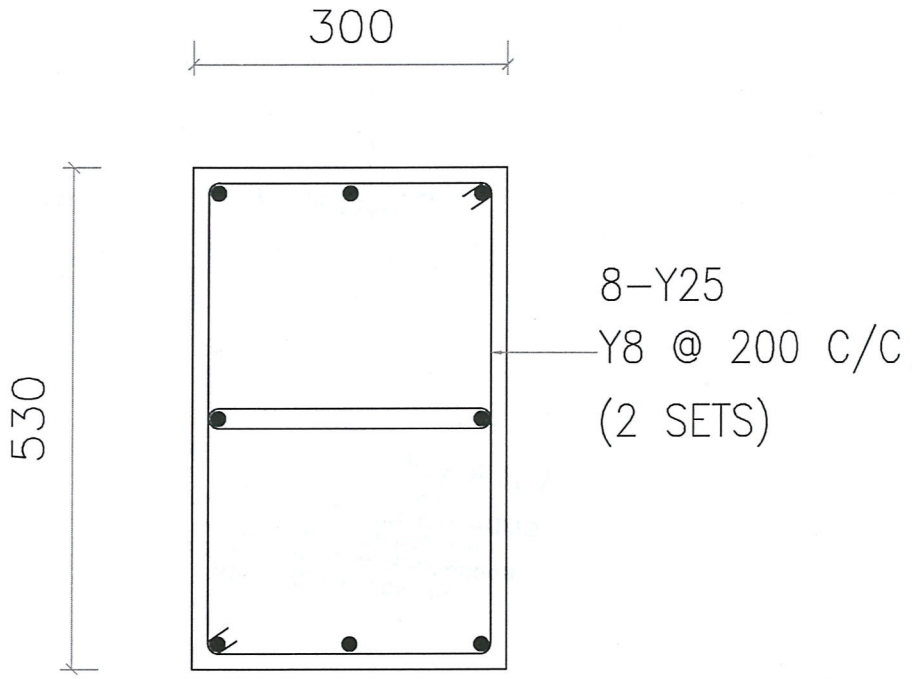
= 14.1

= 13.5 T/m² < 15 T/m²

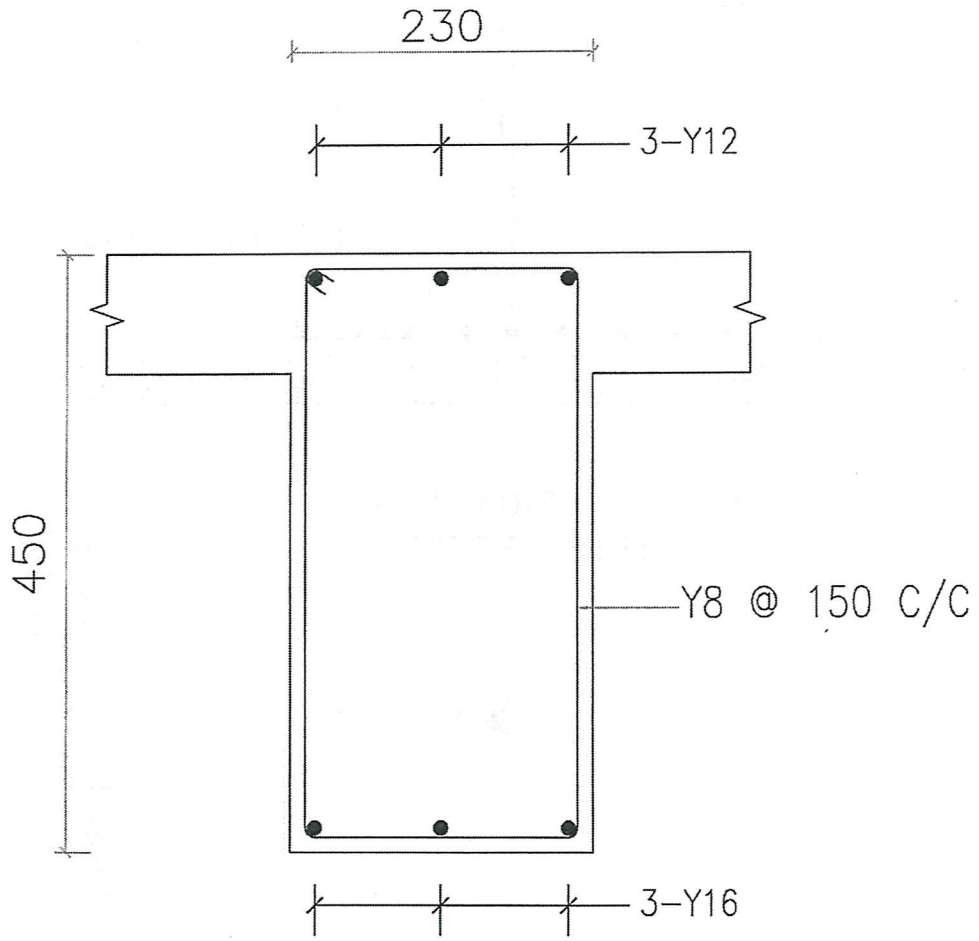
Hence safe

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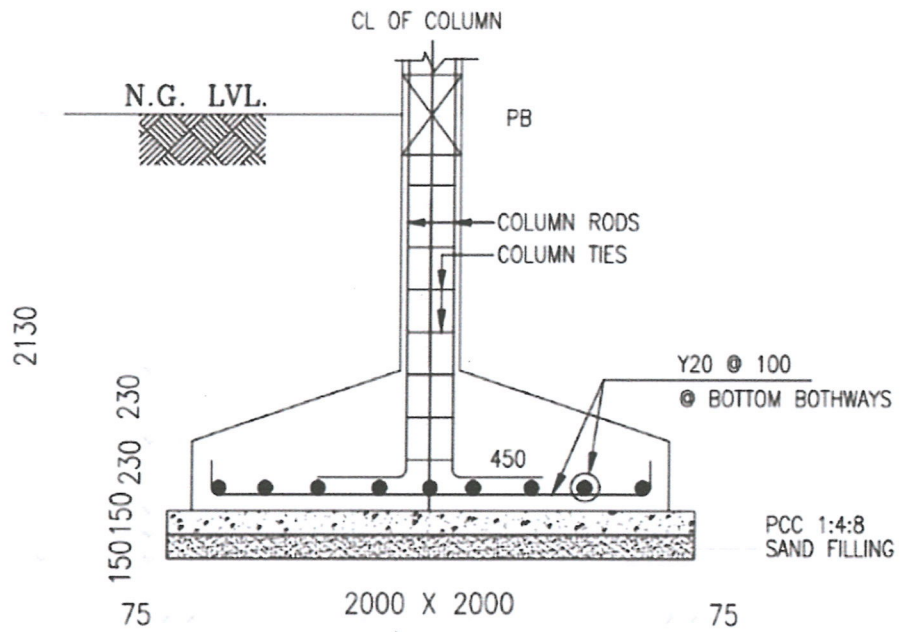
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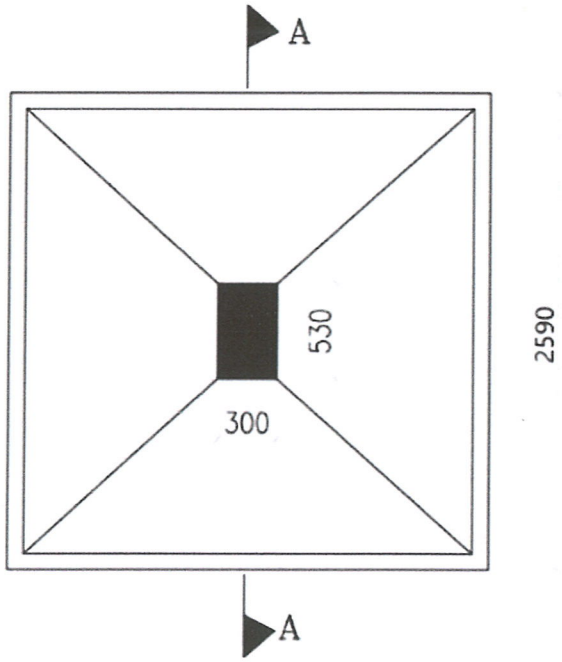
SECTION THRU' COLUMN



SECTION THRU'
FLOOR BEAM

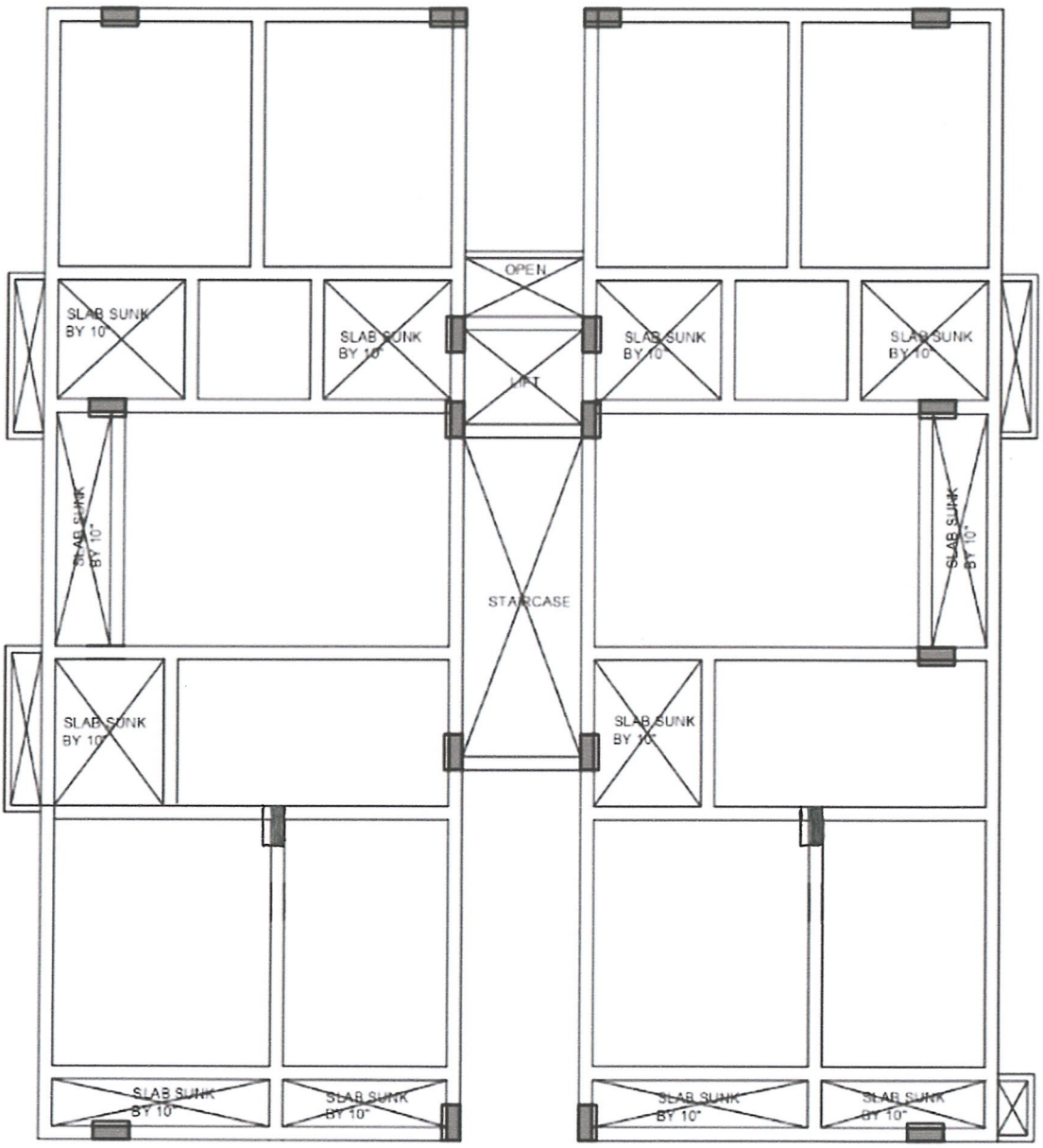


SECTION A–A
THRU' FOOTING



PLAN OF FOOTING

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LAYOUT OF ROOF SLAB & BEAMS OVER STILT FLOOR

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