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DESIGN OF SLAB

Data:

Concrete	M20	fck	=	20	N/mm^2
Steel	Fe500	fy	=	500	N/mm^2
Shorter Span		Lx	=	2.97	m
Longer Span		Ly	=	5.26	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	=	2.97	m
Eff.Longer Span		L_y	=	5.26	m

1.Load Calculations :

Self weight of slab	(0.15 x	20.00)	=	3.00	kN/m^2
Ceiling Plastering			=	0.25	kN/m^2
Floor Finish			=	1.00	kN/m^2
Weathering Course (Weight Not Applicable)			=	0.00	kN/m^2
Live load			=	2.00	kN/m^2
Total Load			=	7.00	kN/m^2
Load Factor			=	1.50	
Factored Load			=	10.50	kN/m^2
Span Ratio		l_y/l_x	=	5.26/2.97	
			=	1.77	

2. Bending Moment Calculation:

Support Condition — Two Adjacent edges discontinuous continuous)

From IS 456-2000 Table 26

αNx	=	0.086
αPx	=	0.065
αNy	=	0.047
αPy	=	0.035

Shorter Span

Negative Moment (support) Mnx	=	$\alpha_{Nx} \times W_u \times L_x^2$
	=	$0.086 \times 10.50 \times 2.97^2$
	=	7.97kN.m
Positive Moment (Mid Span) Mpx	=	$\alpha_{px} \times W_u \times L_x^2$
	=	$0.065 \times 10.50 \times 2.97^2$
	=	6.02kN.m

Longer Span:

Negative Moment (Support) MNy	=	$\alpha_{Ny} \times W_u \times L_x^2$
	=	$0.047 \times 10.50 \times 2.97^2$
	=	4.35kN.m
Positive Moment (Mid span) Mpy	=	$\alpha_{py} \times W_u \times L_x^2$
	=	$0.035 \times 10.50 \times 2.97^2$
	=	3.24kN.m

A.SHANAVAZ M.E., M.I.E., M.I.S.E.,
Chartered Civil Engg. M-150843-6
Chartered Struct Engg. CE/ISE/LM/203
Regd. Struct Engg. Coimbatore LPA
S.E/GR-I/20/01/002

Reinforcement :

$$Mu = 0.87 \times fy \times Ast \times d \left\{ 1 - \frac{(fy \times Ast)}{fck \times bd} \right\}$$

$$= 0.87 \times 500 \times Ast \times 126 \left\{ 1 - \frac{500 \times Ast}{20 \times 1000 \times 126} \right\}$$

Ast in Shorter Span

Support =149mm^2

Mid Span =112mm^2

Ast in Longer Span

Support =110mm^2

Mid Span =90mm^2

	Reinf Required		Reinf Provided
Shorter Span			
Support	8ø @ 340mm c/c	-	8 dia@ 150 mm c/c
Mid Span	8ø @ 320mm c/c	-	8 dia@ 150 mm c/c
Longer Span			
Support	8ø @ 380mm c/c	-	8 dia@ 150 mm c/c
Mid Span	8ø @ 270mm c/c	-	8 dia@ 150 mm c/c

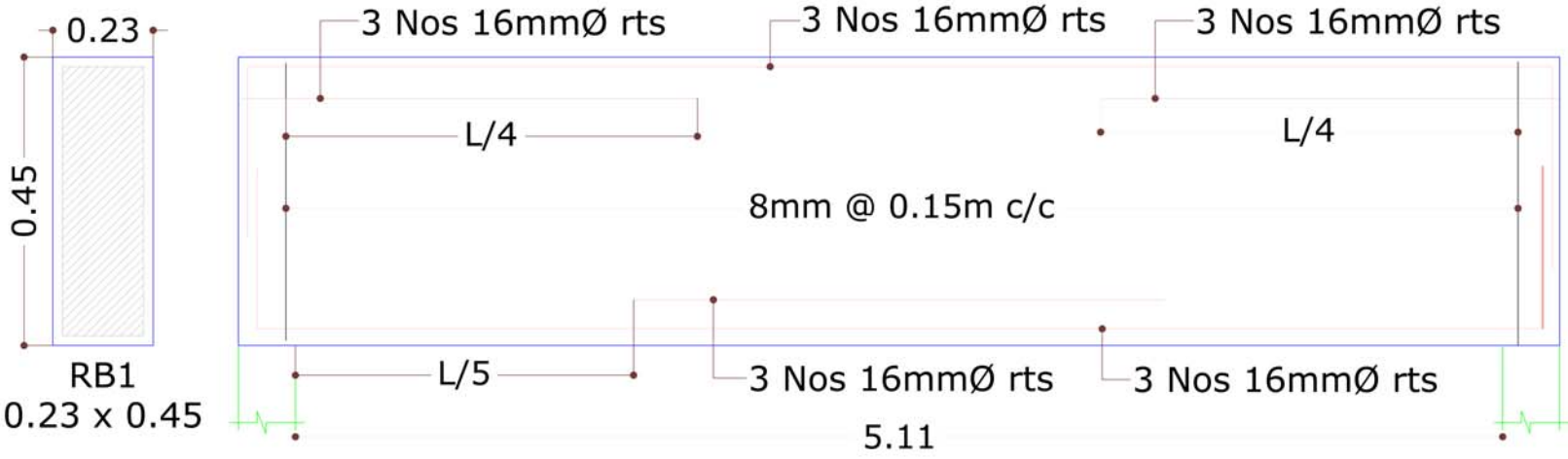
3. Check for deflection

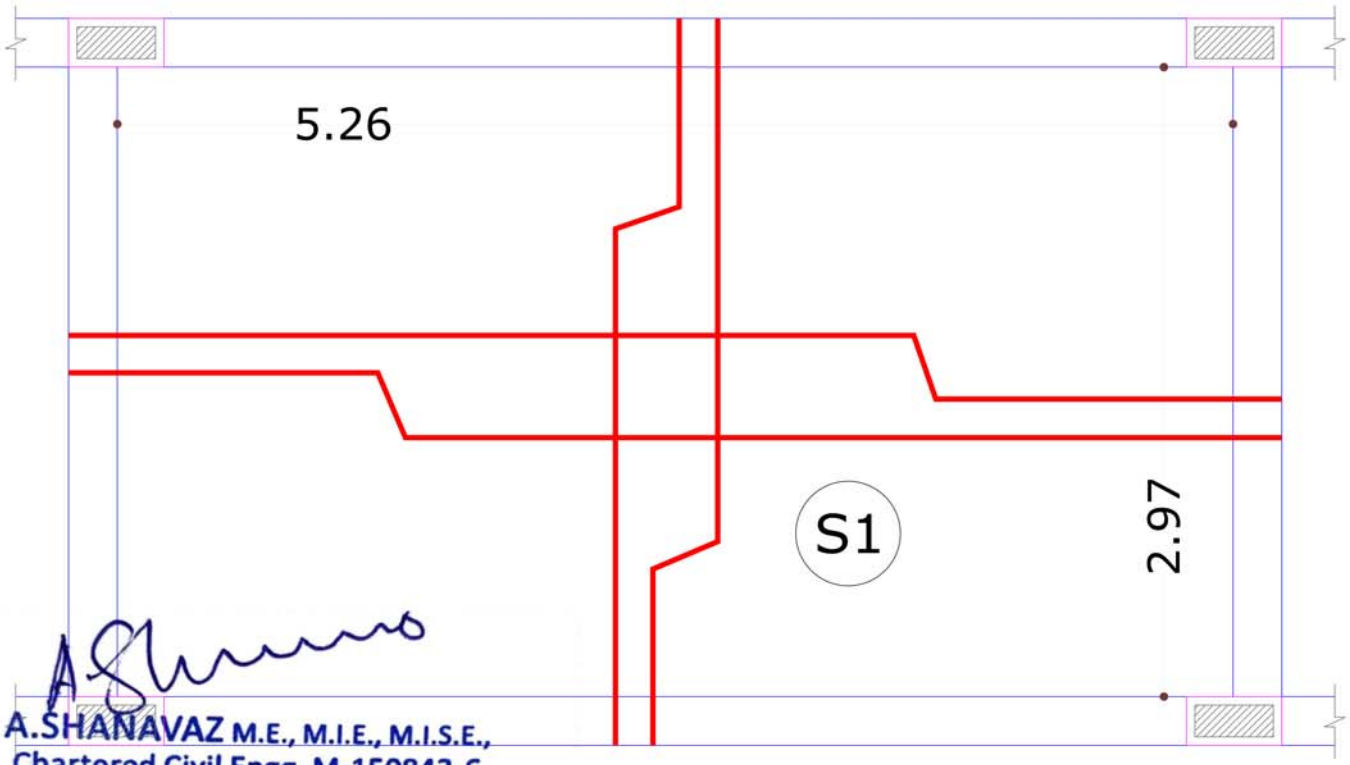
Allowable span / depth Ratio		I/D	=	32
% of Tension reinforcement		pt	=	0.179
Corresponding Compression reinf.		Bar dia	=	8mm
		spacing	=	150mm
% of compression reinforcement		pc	=	0.179
Modification factor for Tension reinf.	F1	=	1.20	(From Fig 4 in IS 456 – 2000)
Modification factor for comp.reinf.	F2	=	1.00	(From Fig 5 in IS 456 – 2000)
Modified span/ depth ratio		=	l/d xF1 xF2	
		=	32 x 1.20 x 1.00	
		=	38.40	
Actual span / depth ratio		=	2970 / 126	
		=	23.57	

Actual span / depth ratio < Modified span /depth ratio 23.57 < 38.40

HENCE SAFE & SOUND

A.SHANAVAZ M.E., M.I.E., M.I.S.E.,
Chartered Civil Engg. M-150843-6
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S.E/GR-I/20/01/002

S.NO	REINFORCEMENT IN SHORTER SPAN	REINFORCEMENT IN LONGER SPAN	SL.TK
S1	8mm Ø 0.15m c/c CKD	8mm Ø 0.15m c/c CKD	0.150m

DESIGN OF BEAM

Data:

Effective length		=	5.11 m
		=	5110 mm
Breadth of Beam		=	230 mm
Overall depth (assumed)		=	450 mm
Characteristic strength of steel			
	fy	=	500 N/mm^2
	fck	=	20 N/mm^2
	Cover	=	25 mm

Total Load

Dead load from slab		=	3.75 kN/mm^2
Ceiling Plastering		=	0.25kN/m^2
Floor finish		=	1 kN/mm^2
Total dead load		=	5.00 kN/m^2
Live load		=	2 kN/m^2
	Total Load	=	7.00 kN/m^2

Effective Depth = d = 450 - 25 - 25 / 2

= 412.5 mm

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

d' = 25 +(25/2)

= 37.5 mm

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

= 0.09

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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 = Characteristic strength of reinforcement

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

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

= 161.29 mm2

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 = $\frac{Wl_x}{6}\left(3 - \left(\frac{lx}{ly}\right)^2\right)$

= $\frac{7 \times 3.2}{6}\left(3 - \left(\frac{3.2}{5.11}\right)^2\right)$

=9.74 kN/m

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

= 1.8 kN/m

Total = 11.54 kN/m

Providing 16 mm dia bars at clear cover 25 mm

Effective cover = 25 + 8

= 33 mm

Effective depth d = 417 mm

Center to center bearing= 5.11 m

Effective span = 5.11 m

Max. BM = $\frac{11.54 \times 3.2 \times 3.2}{8}$

= 14.77 kN.m

Req.depth d = $\left\{\frac{1.5 \times 14.77 \times 10 \wedge 6}{2.75 \times 230}\right\}^{1/2}$

= 187.16 mm

Provided depth, D = 450 mm

$\frac{Mu}{bd^2}$ = $\left\{\frac{1.5 \times 14.77 \times 10 \wedge 6}{230 \times 417 \times 417}\right\}^{1/2}$

= 0.75

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

= 0.18 %

A.SHANAVAZ M.E., M.I.E., M.I.S.E.,
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Ast = $\left\{ \frac{0.18 \times 230 \times 417}{100} \right\}$

= 2884 mm^2

Provided 6 – Y16

Check for Shear

V = $\frac{1.5 \times 11.54 \times 5.11}{2}$

= 44.22 kN

T_c = 0.54 N/mm^2

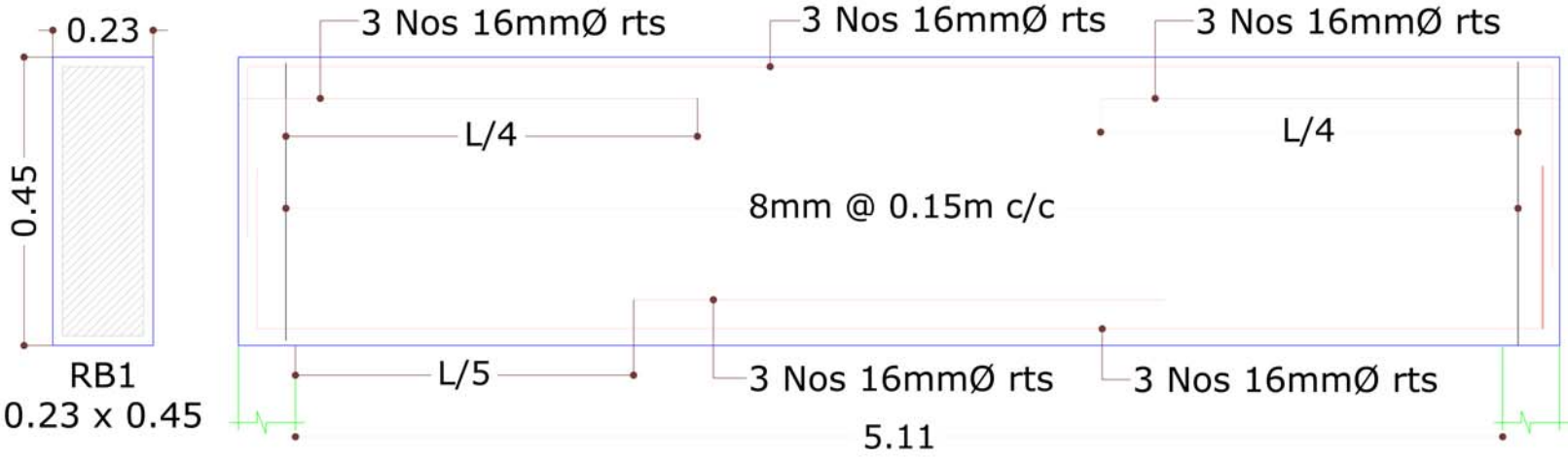
V_{uc} = $\frac{0.48 \times 230 \times 417}{1000}$

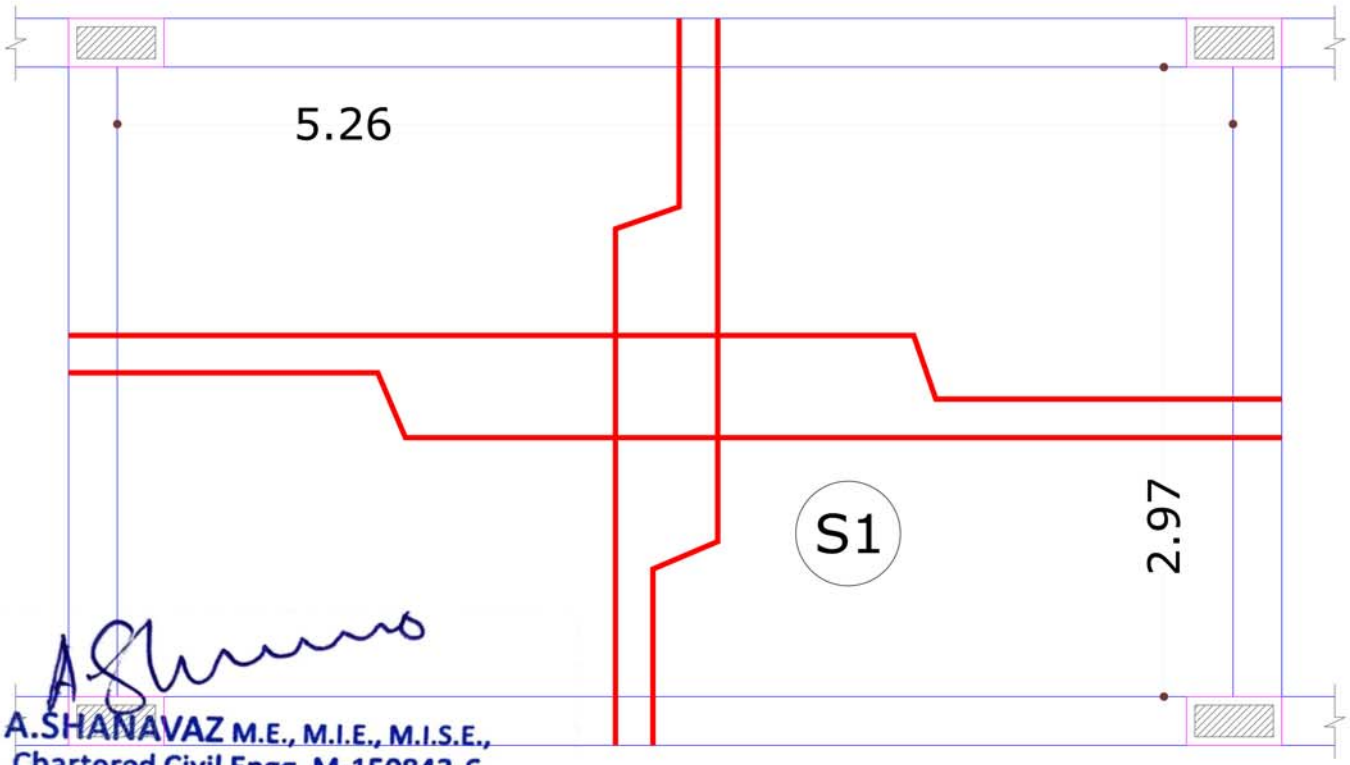
= 46.1 kN

V_{uc} > Shear at support

Provide Y8 @ 150 c / c (stirrups)

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Chartered Civil Engg. M-150843-6
Chartered Struct Engg. CE/ISE/LM/203
Regd. Struct Engg. Coimbatore LPA
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A.SHANAVAZ M.E., M.I.E., M.I.S.E.,
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S.NO	REINFORCEMENT IN SHORTER SPAN	REINFORCEMENT IN LONGER SPAN	SL.TK
S1	8mm Ø 0.15m c/c CKD	8mm Ø 0.15m c/c CKD	0.150m

DESIGN OF COLUMN

Data:

Total No. of floors		=	5
Total No. of Roof		=	6
Storey height		=	3 m
Breadth of column	b	=	230 m
		=	0.23 m
Depth of column	D	=	600 mm
		=	0.60 m
	fck	=	20 N/mm^2

Total axial load

Dead load & live load from one floor = 280 kN

Total axial load all floors = 6 x 280

= 1680 kN

Column dimension = 230 x 600 mm

Self weight of column = 0.23 x 0.60 x 6 x 20 x 3

= 50 kN

Total load = (1680 + 50) x 1.5

Pu = 2595 kN

Area of Reinforcement

$\frac{d'}{D}$ = 0.08

d' = 50 mm

$\frac{Pu}{fck \times b \times D}$ = $\frac{2595 \times 10^3}{20 \times 230 \times 600}$

= 0.95

$\frac{Mu}{fck \times b \times D2}$ = $\frac{1.5 \times 30.48 \times 10^6}{20 \times 230 \times 600^2}$

= 0.02

Refering to chart in SP : 16 for values

$\frac{Pu}{fck.b.D}$ = 0.95 : $\frac{Mu}{fck.b.D^2}$ = 0.02

The value of $\frac{P}{fck}$ = 0.05

P = 0.05 x

= 1.0

A.SHANAVAZ M.E., M.I.E., M.I.S.E.,
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$$As = \frac{p \times b \times D}{100}$$

$$= \frac{1.6 \times 230 \times 600}{100}$$

$$= 2208 \text{ mm}^2$$

Provide 12 – Y16 bars.

Design of Lateral Ties

Dia of lateral ties = $\frac{1}{4} \times 16 = 4$ mm

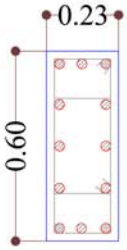
Hence 8mm dia bar is adopted

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

a. Least lateral dimension = 300 mm
b. 16 x dia of bar
c. 300 mm

Hence provide Y8 @ 150mm c/c
Similarly other columns are calculated.

A.SHANAVAZ M.E., M.I.E., M.I.S.E.,
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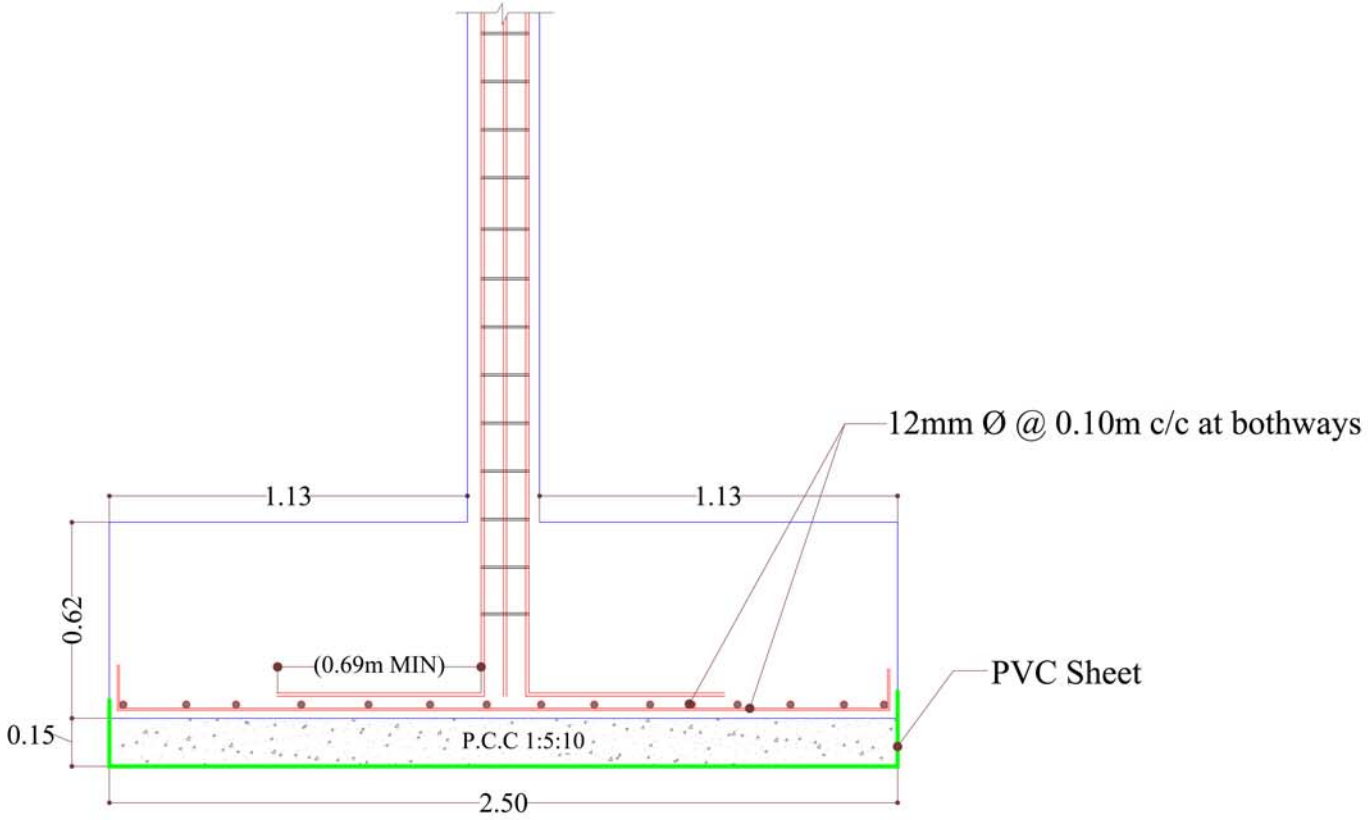


Main Reinforcements
12 Nos 16mm Ø rts
4 Legged Stirrups
8mm @ 0.15m c/c

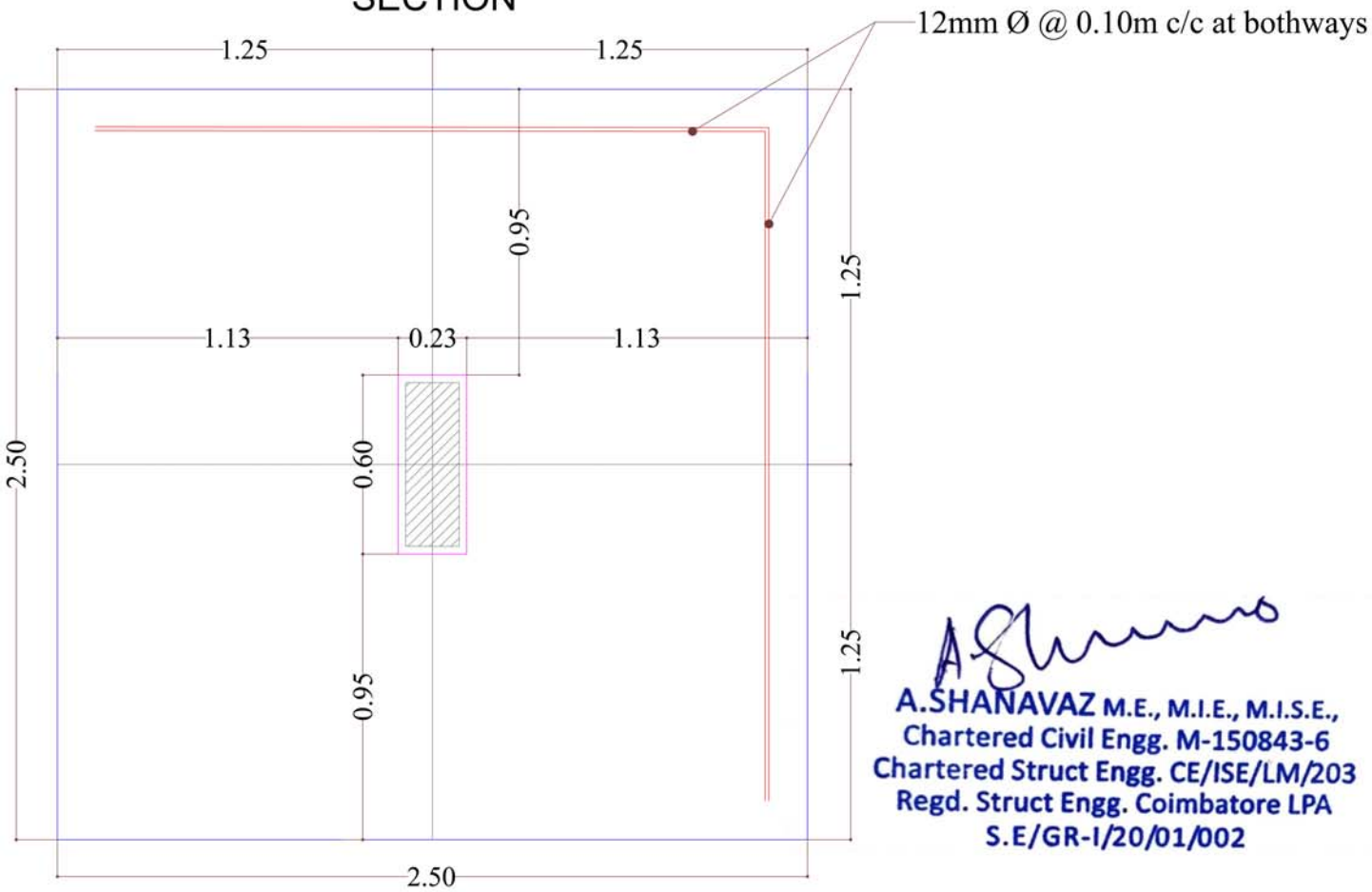
COLUMN - C1
(0.23 x 0.60)



4 Legged Stirrups
8mm @ 0.15m c/c



SECTION



PLAN

A.SHANAVAZ M.E., M.I.E., M.I.S.E.,
Chartered Civil Engg. M-150843-6
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DESIGN OF FOUNDATION

Design of Isolated footing for Intermediate column

Data :

Column Size	L	=	230 mm
	B	=	600 mm
Size of footing	L	=	2.5 m
	B	=	2.5 m
Safe Bearing capacity of soil	SBC	=	450 kN/m^2
Depth of footing below GL		=	2 m
Density of soil	g	=	18 kN/m^3
Characteristic strength of concrete	fck	=	20 N/mm^2
Characteristic strength of Steel	fy	=	500 N/mm^2
Depth of footing provided	D	=	1.2m
Cover	d'	=	50mm

1. Loading on footing

Axial load on column = 2595 kN

Self weight of footing (10%) = 131 kN

Total load on footing = 2726 kN

Adpot size of footing = 2.5 x 2.5 m

= 6.25 m^2

Soil pressure $\frac{P}{A} = \frac{2726}{6.25}$

= 436 kN/m^2

Gross safe bearing capacity = 450 kN/m^2

Hence safe & sound

2. Moment Calculation

Face of the column = $\frac{2.5 - 0.60}{2}$

= 0.95 m

Design pressure = 436 kN/m^2

Moment M = $\left\{ \frac{436 \times (0.95)^2}{2} \right\}$

= 196.75 kN.m

Factored moment = 295.12 kN.m

A.SHANAVAZ M.E., M.I.E., M.I.S.E.,
Chartered Civil Engg. M-150843-6
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3. Area of Reinforcement

Effective Depth d = 620 mm (or) 0.62 m

Dia of bar provided ø = 20 mm

Provided spacing = 100mm

$$\frac{Mu}{b \times d^2} = \frac{295.12 \times 10^6}{1000 \times 620 \times 620}$$

$$= 0.77$$

Percentage of tension reinforcement

$$pt = \frac{fck}{2 \times fy}\left\{1-\left(\text{SQRT}\left[1 - \frac{4 \times (Mu/b \times d2)}{0.87 \times fck}\right]\right)\right\} \times 100$$

$$= 0.51\ \%$$

Required, Ast $= \frac{0.51 \times 1000 \times 620}{100}$

$$= 3162\ mm^2$$

Hence safe & sound

4.Check for One way shear

Vu = 1.5 x 254 x 3 x 0.5 x 0.30 x 0.5 – 0.62

= 160 kN

Permissible shear stress $= \frac{160 \times 1000}{1000 \times 620}$

$= 0.26\ N/mm^2$

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

Hence safe & sound

5.Check for punching shear

Punching force Pf $= \left(254 \times 608 - ((0.23 + 0.62) \times (0.60 + 0.62))\right) \times 1.5$

= 2187 kN

Punching shear stress $= \frac{2{,}187 \times 1000}{620 \times 2 \times (230 + 6000 + (2 \times 620))}$

$= 0.85\ N/mm^2$

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

Hence safe & sound

EARTH QUAKE ANALYSIS

1. Calculation of horizontal seismic Coefficient

As per IS 1893 – 2002

$$Ah = \frac{z \times 1 \times Sa}{2 \times R \times g}$$

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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 = $0.075\ h^{3/4}$
(Clause 7.6) h = height of the building

$= 0.075 \times 13.1^{0.75}$

$= 0.52$ sec

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

$= 0.04$

2. Calculation of Base shear & seismic forces

Base shear V = $Ah \sum Wi$

$= 0.040 \times 1316$

$= 52.64$ kN

Total no of columns = 21

Shear per column = $\frac{25.64}{21}$

Seismic moment per column = $\frac{1.22 \times 3}{4}$

$= 0.92$ T-m

Column provided = 230 x 600 mm with 12 –Y16

Hence safe & sound

3. Checking for footing

SBC = $45\ T/m^2$

Soil pressure = $25.4\ T/m^2$

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

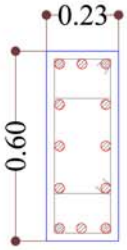
$= 25.4 \pm 0.7$

$= 26.1$

$= 24.7\ T/m^2 < 45.00\ T/m^2$

Hence safe & sound

A.SHANAVAZ M.E., M.I.E., M.I.S.E.,
Chartered Civil Engg. M-150843-6
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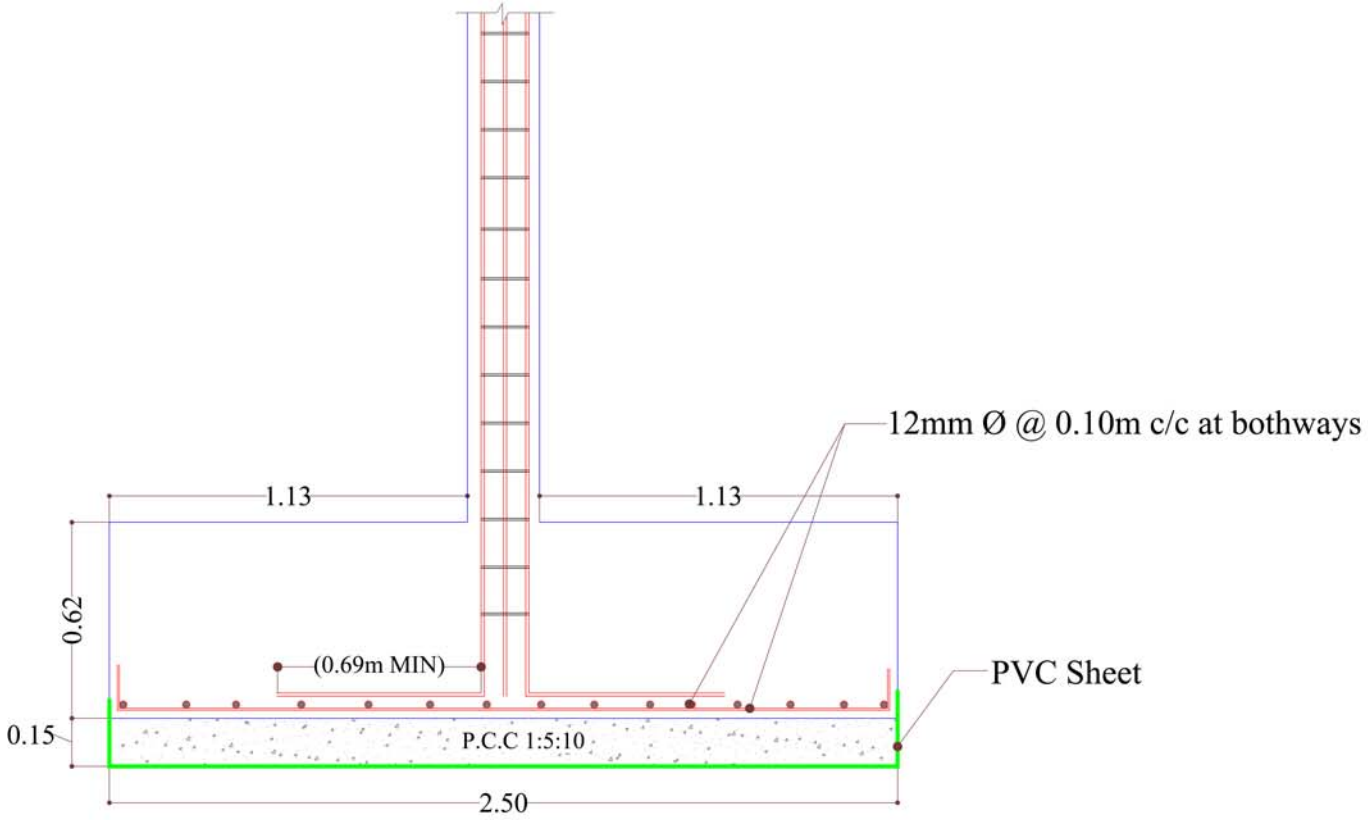


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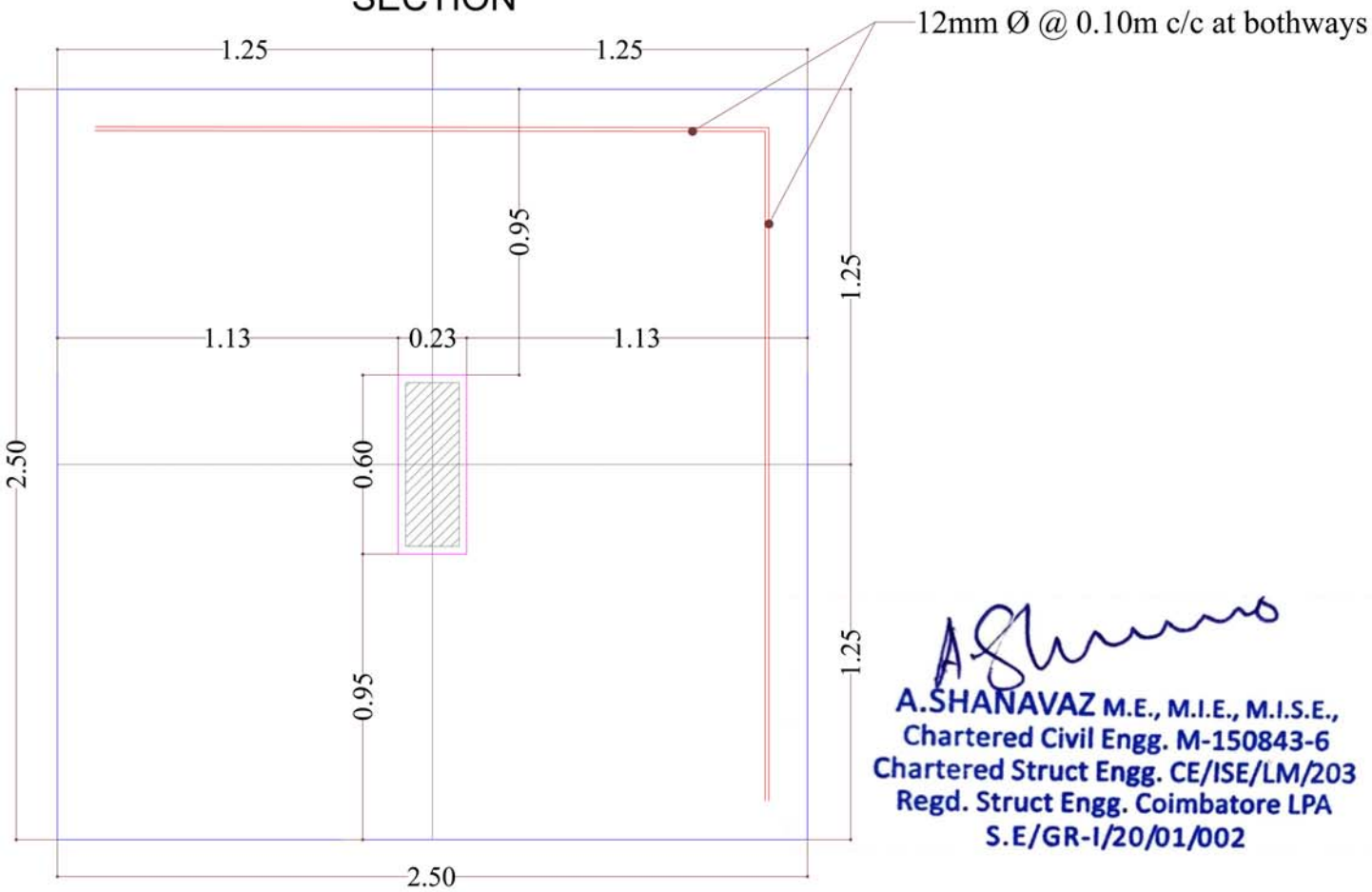
COLUMN - C1
(0.23 x 0.60)



4 Legged Stirrups
8mm @ 0.15m c/c



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A.SHANAVAZ M.E., M.I.E., M.I.S.E.,
Chartered Civil Engg. M-150843-6
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Regd. Struct Engg. Coimbatore LPA
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hello@verified.realestate
+91 9996 12 9996
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